Studying the causes of the open type overland car parking collapse in Nur-Sultan

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Abstract. Studying the causes of accidents allows avoiding mistakes in the design, construction and operation of the building. According to the results of a continuous visual and detailed instrumental examination, the materials are collected and analyzed in the article that characterize the causes of the accidental collapse of the overlapping section of the metal frame. The calculation of steel beams of the overlapping section in the “2-3/B-V” axes at the +4,500 mark of the overland parking building was performed in the LIR-SCC design system of the LIRA-CAD 2013 program both for design combinations of efforts and for design combination of loads. Design errors that led to the accident are identified. Both the results of the calculation of the bolted connection, and the calculation of steel profiled flooring are presented. In addition, technical decisions are made on the further safe operation of the facility. This accident draws attention to the problem of ensuring safety of buildings and structures and the need for an integrated approach in the framework of industrial safety expertise.

1. Introduction
One of the main tasks of construction is to ensure reliability of the structure. In the design of new buildings, as well as in the reconstruction of existing ones, it is necessary to meet present day criteria of reliability and safety. In the analysis of the causes of emergency collapses of buildings and structures, special attention should be paid to studying the factors that lead to an emergency condition or collapse of load-bearing building structures. A deep, objective analysis of the causes of the accident will help to avoid further errors in the design of buildings and structures, construction, as well as during their operation.

During the operation of structures, visual inspection methods are widely used to assess the technical condition of a building. The procedure of checking the condition of the building is described in work [1]. If serious problems are found, special analysis can be required. In article [2], new trends of research in the field of mechanical safety and survivability of structures of buildings and structures under various loads and impacts, including regular and random types of loads, are examined.

To establish the level of design reliability, various control methods are used. A qualitative assessment of reliability of buildings and structures allows predicting the likelihood of an accident. So in article [3], risk analysis methods of assessing the technical condition of buildings and their structures are considered. Articles [4, 5] propose an approach to assessing the risk of building collapse with the damage localization. Studies [6, 7] provide an analysis of buildings safety as a result of high loads. The examples are given of improper construction and maintenance.
According to the statistics of studying buildings and structures for civil and industrial purposes, it was found that the main causes of the emergency are as follows [9]:

- deviation from design decisions inconsistent with the design organization during construction and mounting work;
- low quality materials that are used in the construction, repair or reconstruction of facilities;
- poor-quality manufacturing of structures;
- low quality of construction and mounting work;
- errors in the design process.

As an argument, we can give an example of an emergency collapse of the overlap section under construction of an open-type parking for cars in the city of Nur-Sultan (hereinafter referred to as the Object). The collapse area between four load-bearing columns was 133.21 m² (see figure 1).

![Figure 1. General view of the collapse.](image)

2. Methods
The Parking building is a steel frame consisting of columns and beams (crossbars) of a continuous I-section.

The layout of the Object is a five-story rectangular building with dimensions in the rows “A-B” – 29.1 m and in the axes “AB” – 139.6 m, with the main platforms at the first span mark in the axes “AB” – 0.250; +3.000; +6.000; +9.000; +12.000; +14.700; and the second span in the B-V axes – 0.250; +4500; +7.500; +10.500; +13.500; +16.200.

The ceilings are made of reinforced concrete on a non-removable formwork of a profiled sheet supported by longitudinal welded I-beams and transverse split welded beams made of a two-tee with changing the cross-section.

Longitudinal welded steel I-beams of continuous cross-section are made of rolled sheets: beam shelves of rolled sheets – 240x12, beam walls of rolled sheets – 600x8 mm. Cross-sectional welded steel beams of continuous cross-section are made of rolled sheets: beam shelves of rolled sheets – 240x12, beam walls at the points of support of rolled sheets – 400x8, in the middle of the beam – 580x6. Reinforcement of monolithic floor slabs is made of reinforcement $\phi 12$ AIII with a pitch of 200 mm.

Based on the results of a continuous visual and detailed instrumental examination, the following materials were collected and analyzed that characterize the causes of the emergency collapse of floor beams [10-12] at +4.500.
1. The nature of the concrete mass descent (see figure 2) indicates that the right side (when viewed from the facade of the structure) of B-43 main beam collapsed in the first place. At the same time, the secondary beam B-26 received torsional deformation (see figure 3), due to the transfer of the tensile forces of the reinforcing mesh and the gravity of the descending concrete masses to the upper shelf of the beam, and the secondary beams of the adjacent span received a “saber” deformation (see figures 4, 5), partially compensating thereby a part of the total torsional strain energy perceived by beam B-26.

![Figure 2. The nature of the concrete mass descending with the ceiling beams collapse.](image)

2. The confirmation that the right side of B-43 main beam collapsed in the first place is also the torsion strain (counter deformation, see figure 3) of B-27 beam received by it due to the rotation of the right side of B-43 beam relative to its (still attached) left side. Due to the impact that arose in the beam from shock contact with the ground and at the same time, the fastening bolts of the left side of the beam were cut off (due to a turn), the beam collapsed completely.

![Figure 3. Counter torsional deformation of the secondary B-26 beams (left) and the ceiling beam B-27 (right).](image)

![Figure 4. “Saber” deformation of the secondary beams from the bending plane.](image)
3. It can be seen in Figures 6-10 that the slot holes in the overlay plates are Ø18mm and 30mm long (by the way, this contradicts the regulatory tolerance for beam slope), in the plant they are made with a gas cutter (as evidenced by the presence of a primer) with an insufficient degree of step accuracy holes and low quality cutting edges. Obviously, the project implied milled hole making, that the plant was not able to perform due to its insufficient technological equipment.

When mounting B-43 main beam, due to the unacceptable “blackness” of the holes of the assembled mounting package, the slot holes of the beam wall were additionally extended, as evidenced by the color of the tarnish from gas cutting.

The result of this assembly mounting was that the initial load from B-43 main beam on the right and left sides was perceived by one fixing bolt (on each side).

It can be seen in Figures 6-10, in fact that the load from the beam in the left and right nodes (up to the collapse) was perceived in total by two bolts, instead of the five bolts indicated by the project.
4. It can be seen in Figure 11 that the holes for the structural bolts for securing the secondary beam B-26 are also made by gas cutting. It should be noted that the bolts of B class accuracy with the diameter of 16 mm, it is recommended to be mounted in the d = 19 mm holes instead of 20 mm made. For shear joints, the difference between the diameter of the body of the bolt and the hole should not exceed 3 mm. When monitoring the state of the fastening patch plates, it was found that the tension of the bolts did not match the design value, and that the upper edge of the left plate was deformed [13-15].
It is obvious that the deformation of the plate occurred at the initial stage of collapse, due to the elastic-plastic deformation of the upper flange and the wall of the main B-43 beam (see figure 12), which in turn was caused by insufficient rigidity of the secondary floor beams B-27, B-28.

3. Results and Discussion

Laboratory studies of M16 bolts of the 4.8 strength class indicate that the bolts are made of steel grade 10, GOST 1050-88 and have the following mechanical characteristics $[\sigma_u] = 602$ MPa, $[\tau] = 325$ MPa, $HB = 210$-245, $\delta = 13\%$.

The standard mechanical characteristics that a bolt of the 4.8 strength class, according to GOST 1759.4-87, must meet are as follows: $[\sigma_u] = 400$-420 MPa, $[\tau] = 320$-340 MPa, $HB = 124$-238, $\delta = 14\%$.

The comparative analysis indicates that the bolts of this batch of manufacture have strength (ultimate resistance limit) by 200 MPa larger than their standard value. With the indicated value of tensile strength corresponding to the value of yield strength and at the same time increasing hardness (on average by 46HB) with decreasing the standard value of relative elongation (1%), it can be stated with full certainty that these bolts were subjected to heat treatment.
It should be noted that for fastening the beams to the columns, the project indicates M20 bolts of the 5.8 strength class (5.6) according to GOST 1759.4-87 of the following design (see figure 13).

![Figure 13. M20 bolts of the 5.8 (5.6) strength class allowed in the design by GOST 1759.4-87.](image)

In fact, when mounting, M16 bolts of the 4.8 strength class with a full threaded portion b and non-design length L were used (see figure 14). As a result, the bolt cut occurred along the threaded weakened part of the rod.

![Figure 14. General view of the actually made bolts.](image)

The calculation of profiled steel flooring at the stage of erection of the floor showed the following. According to the results of an instrumental examination, it was found that for a monolithic overlap in the “2-3/B-V” axes at the + 4.500 mark, a fixed formwork is steel profiled flooring of the H57-750-0.8 grade (see figure 15). Physical-and-geometric characteristics of the flooring are presented in Table 1.

Table 1. Characteristics of the H57-750-0.8 grade floor.

| Cross section dimensions, mm | Cross section A, cm² | Inertia moment, Iₓ, cm⁴ | Resistance moment Wₓ, cm³ | Mass of 1 m², kg |
|-----------------------------|----------------------|-------------------------|--------------------------|-----------------|
| h  B₁   t   B   b   b₁   b₂   b₃   b₄   b₅   h₁, not less h₂, R, no more S | 801 | 94.5 | 44 | 42 | 20 | 93 | 46.5 | 18 | 10 | 7 | 4 | 187.5 | 8.8 | 18.9 | 24.0 | 9.8 |
Figure 15. Profiled sheet of the H type with the height of 57 mm.

According to paragraph 4.3 of the “Recommendations for the design of monolithic reinforced concrete floors with profiled steel flooring”, the strength conditions of the flooring are as follows:

\[
\frac{M}{W_x} \leq R_n, \quad \frac{Q}{t h_n} \leq R_n
\]

where \( W_x \) is the resistance moment with wide compression flanges, GOST 24045, equal to 24 cm³;
\( h_n \) is the height of the steel profiled floor;
\( t \) is the thickness of the steel profiled floor;
\( Q \) is the transverse force equal to 1.226 \( t \);
\( R_n = 2400 \text{kI/cm}^2 \) is the calculated floor steel resistance.

The value of the maximum bending moment per 1 m of the width is:

\[
M = \frac{q i^2}{8} = \frac{956 \cdot 2.57^2}{8} = 789 \text{t cm} = 789000 \text{kg cm}
\]

From here

\[
\frac{M}{W_x} = \frac{78100}{24} = 3254 \text{kg/cm}^2 > 2400 \text{kg/cm}^2
\]

\[
\frac{Q}{t h_n} = \frac{1226}{0.456} = 2689 \text{kg/cm}^2 > 2400 \text{kg/cm}^2
\]

The condition is not satisfied, the profiled sheet strength is not provided.

According to paragraph 4.7 of the “Recommendations for the design of monolithic reinforced concrete floors with profiled steel flooring”, the deflection of the flooring at the construction stage is determined by the formula:

\[
f_n = k_n \left( \frac{q^n \cdot i^4}{E_{sg} I_{sg}} \right) + a \leq \frac{1}{200} i_n
\]

where \( q^n = 760 \text{kg.m.m} < 0.00746 \text{MN-m} \);
\( E_{sg} = 2.06 \cdot 10^5 \text{MPa} \) is the steel elasticity modulus;
\( I_{sg} \) is the beam inertia moment “n”;
a = 2 \text{ mm} is an empiric value for a multi-span floor;
\( k_n \) is the coefficient determined depending on the floor spreading, 0.0091.

\[
f_n = 0.025 > 0.013
\]

The condition is not satisfied, the floor rigidness is not provided.
The calculation of the bolt-type connection is made according to paragraph 11.7-11.8 of RK SNIP 5.04-23-2002 “Steel structure. Norms of designing”.

The bolt bearing capacity from the condition of shear strength of the bolt body:

\[ N_{bs} = R_{bs} \cdot A \cdot \gamma_b \cdot n_i \]

Where \( R_{bs} \) is the calculated shear resistance of the bolt connection; 
\( A \) is the calculated bolt cross section; 
\( \gamma_b \) is the coefficient of the bolt connection operation; 
\( n_i \) is the number of calculated bolt cuts.

The bolt bearing capacity from the condition of the hole walls collapse resistance:

\[ N_{bp} = R_{bp} \cdot \gamma_b \cdot d \cdot \sum t \]

\( R_{bp} \) is the calculated collapse resistance of the bolt connection; 
\( d \) is the outer diameter of the bolt rod; 
\( \sum t \) is the least total thickness of the elements collapsed in one direction.

From here the required number of bolts for the connection:

\[ n \geq \frac{1.2 \cdot N}{\gamma_c \cdot N_{min}} \]

\( n \) is the required number of bolts; 
\( N \) is the longitudinal force; 
\( \gamma_c \) the coefficient of the operation condition; 
1.2 is the coefficient considering the section turn on the support and eccentric load applying; 
\( N_{min} \) is the least of the values of calculated effort for one bolt.

According to the project, a bolted connection with the bolts of normal accuracy with the diameter of \( d = 20 \) mm, strength class 5.6, was adopted. However, bolts with the diameter of \( d = 16 \) mm, strength class 4.8, were mounted in BP2 beam and column K connection. The calculation results for the required number of bolts \( n \) for the three calculation options are presented in Table 2.

### Table 2. Results of the bolted-type connection.

| Option number | \( N \) (kg) | bolts \( d = 20 \) mm of the 5.6 class | bolts \( d = 16 \) mm of the 4.8 class | \( n \) |
|---------------|-------------|-------------------------------------|-------------------------------------|------|
| 1             | 18882       | 10739                               | 6624                                | 3    |
| 2             | 15133       |                                     | 6624                                | 4    |
| 3             | 22141       |                                     | 6624                                | 5    |

In addition, according to paragraph 4.31 of RK SNIP 5.04-18-2002 “Metal structures. Rules for the production and acceptance of work”, the thread of the bolts should not go deeper into the hole more than half the thickness of the extreme element of the package on the nut side. Examinations showed that the plane of the cut bolts passed along its threaded part, where the cross section of the body of the bolt was weakened by thread.

Therefore, the bearing capacity of the bolt from the condition of shear strength along the weakened section of the bolt body and without taking into account the stress concentration will be equal to:

\[ R_{bs} = 2 \cdot 1.57 \cdot 1600 \cdot 0.9 = 4522 \text{kg}. \]

From here the required number of bolts for connection:

\[ n = \frac{1.2 \cdot 22141}{4522} = 5.88 \approx 6. \]
Instead of five Ø16 bolts of the 4.8 class that were mounted actually. The bolt-type connection strength is not provided, since by the results of calculations the required number of bolts is larger than in fact.

The calculation of steel beams of the overlapping section in the “2-3/B-V” axes at the + 4,500 mark of the overland parking building was performed in the LIR-STC design system of the LIRA-CAD 2013 program both for design combinations of efforts and for design combination of loads.

According to the calculation results of steel beams of the overlapping section in the “2-3/B-V” axes at the + 4,500 mark of the open-type parking building for cars in the city of Nur-Sultan, there was established the following:

1. The bearing capacity of the main beams of the BP and BP2 grades is not provided for any scheme of the temporary load applying.

2. The bearing capacity and serviceability of the secondary P-type beams is not provided for any scheme of the temporary load applying.

4. Conclusion
Thus, analyzing the results of continuous visual, detailed instrumental, laboratory studies and verification calculations, it was found that the main causes of the accident of the overlap area at the + 4,500 mark in the “2-3/B-V” axes were as follows:

- design errors (profiled flooring, sections of bearing and secondary beams are incorrectly selected);
- uneven load distribution between the bolts of the connection, due to poorly made holes for mounting bolts in the walls of the beam;
- deflections of the secondary B-27 and B-28 beams significantly exceed the maximum permissible standard values, due to their insufficient bending stiffness, which is confirmed by a verification calculation;
- making connections of beams with columns when using non-designed M16 bolts with the 4.8 strength class, against the designed ones, M20 of the 5.8 strength class;
- loss of stability of the bearing beams as a result of gross errors made in the calculation and selection of beam sections.

Among the additional causes that led to the emergency situation of the overlap area at the + 4,500 mark in the axes “2-3/B-V” there was:

- insufficient bearing capacity of the profiled sheet, the main and secondary floor beams.

The accident described draws attention to the problem of ensuring safety of buildings and structures and the need for an integrated approach in the framework of industrial safety expertise for the passage of design documentation, as well as, the increasing responsibility of designers and tightening acceptance control, both during construction and during operation of facilities.

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