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THE INFLUENCE OF TEMPERATURE ACTIONS ON THE CRACK RESISTANCE OF LOAD-BEARING STRUCTURES IN A CAST-IN-SITU BUILDING DURING CONSTRUCTION

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Abstract

This study empirically assesses temperature effects on load-bearing systems using field data from an ongoing multifunctional complex featuring cast-in-situ reinforced concrete framing. The calculation-analytical method was employed for design justification, along with mathematical modeling using the LIRA 10.12 software. The results revealed that the strength utilization factor, considering the design reinforcement, exceeded 100% by up to 200% in certain sections of the 2nd underground floor slab, and ranged from 105% to 200% in sections of the 1st underground floor slab. Based on the results of the research, the following conclusions were drawn: cracks in the load-bearing structures of floor slabs and external load-bearing walls of the -2nd and -1st underground floors occurred due to the insufficiency of the calculated reinforcement for the perception of all types of impacts, including temperature; the main reason for the formation of cracks is the absence of expansion joints in the design document of load-bearing structures of the -2nd and -1st floors. According to the research findings the following recommendations are given: when designing cast-in-situ reinforced concrete frame buildings it is necessary to perform a temperature calculation; in case of failure to perform the calculation, it is necessary to arrange expansion joints per the code recommendations; the use of expansion joints in design can be avoided only with appropriate justification.

Keywords: Cast-in-situ Frame Building, Cracks, Expansion Joint, Temperature Actions, Temperature Deformations, Temperature Shrinkage Block.

Nomenclature

Δ_t free temperature displacement of the element resulting from the temperature effect (shortening or lengthening of the element), m

E modulus of elasticity of material, MPa

Δt_w, Δt_c values of temperature difference across the element cross-section in warm and cold seasons, respectively, °C

t_w, t_c standard temperature values for warm and cold seasons, respectively, °C

t_0 system closing temperature, °C

t_{0w}, t_{0c} initial temperature (closing temperature) in warm and cold seasons, respectively, °C

t_I,t_{VII} long-term average monthly air temperature in January and July, respectively, taken for the above-ground part of the structure, °C

L length of the structural element, m

Greek Symbols

α coefficient of linear expansion (shortening), °C

σ stresses in the element, MPa

ε relative strains of the element, dimensionless

I. Introduction

Nowadays, the structures of cast-in-situ reinforced concrete flat slabs are widely used for buildings of different purposes, such as office and residential buildings, parking buildings, shopping malls, etc. [XII, XX].

Following the requirements of Russian design standards, cast-in-situ buildings should be divided into temperature-shrinkage blocks using expansion joints to prevent uneven settlements and deformations of load-bearing structures [X, XX]. There are several types of joints: temperature, shrinkage, anti-seismic, and expansion-settlement joints [XXIV]. The expansion joint is used to divide the building from the bottom of the foundation to the roof, creating blocks. The combination of four features can result in a multifunctional temperature-shrinkage and settlement-seismic joint [XXIV, XXIII].

The main purpose of expansion joints is to effectively eliminate stresses that arise due to certain factors. These stresses can occur in the cross-sections of reinforced concrete elements due to unfavorable temperature changes, moisture exposure, swelling of structures due to increased humidity during construction, ground movements in earthquake-prone areas, moisture-induced settlement of weak soil bases, and the presence of internal cavities such as karst caves, mine workings, subway tunnels, or technological engineering systems [X, XII, XXI].

For example, in the central part of Russia, northern Europe, the USA, Canada, and other countries and regions, the outdoor air temperature can vary from -30 °C to +30 °C depending on the season. That is, the temperature difference Δt_w can be 60 ° C and higher [II, VII, VIII].

The issues of calculation of buildings and structures for temperature actions have been considered by various scientists since the beginning of the 20th century [I, XV, XXII]. Recommendations on expansion joint spacing from various authors and sources are presented in Table 1 and Figures 1-2.

Table 1: Expansion joint spacings

Author	Spacing
Lewerenz (1907)	75 ft (23 m) for walls.
Hunter (1953)	80 ft (25 m) for walls and insulated roofs, 30 to 40 ft (9 to 12 m) for uninsulated roofs.
Billig (1960)	100 ft (30 m) maximum building length without joints. Recommends joint placement at abrupt changes in plan and at changes in building height to account for potential stress concentrations.
Wood (1981)	100 to 120 ft (30 to 35 m) for walls.
Indian Standards Institution (1964)	45 m (≈ 148 ft) maximum building length between joints.
PCA (1982)	200 ft (60 m) maximum building length without joints.
ACI 350R-83	120 ft (36 m) in sanitary structures partially filled with liquid (closer spacings required when no liquid present).

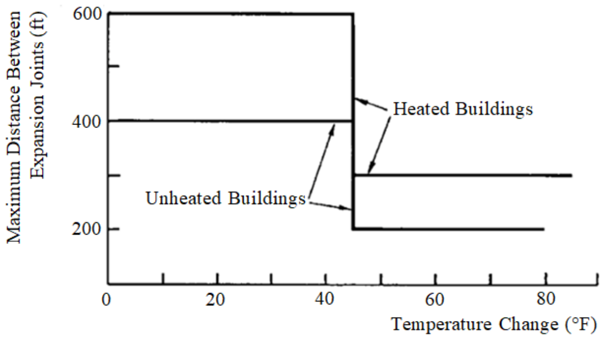

Fig. 1. Expansion joint criteria of the Federal Construction Council (National Academy of Sciences 1974)

Although deviations from standard practice may occur in these recommendations, they remain applicable under certain structural contexts. Determining factors involve the concrete mix design and placement techniques, inherent structural characteristics, the level of restraint imposed on individual elements, as well as the nature and intensity of environmental exposures and service loads.

Contemporary structural engineering lacks a consensus approach to managing deformations arising from thermal and moisture gradients. In this context, limiting building length via prescribed joint spacings, often based on experience, is a common design strategy. Therefore, the problems of expansion joints and calculations for temperature actions are very relevant and in demand for modern design engineers [XII, XVIII].

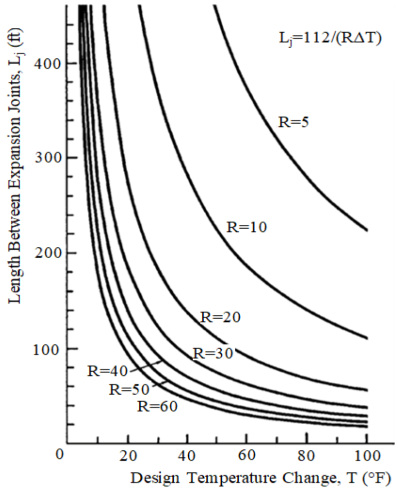

Fig. 2. Length between expansion joints versus design temperature change, AT (Martin & Acosta 1970)

At different times in Russia and abroad, such authors as Aleksandrovsky S.V., Korsun V.I., Mkrtychev O.V., Snegirev A.I., Nguyen, T.C., Bui, K.A., Hoang, Q.L., and many others have studied temperature actions taking into account various factors [I, XVII, XXII, XXIV, XXV]. Sabirov R.A. and Chepurnenko A.S. were engaged in the calculation of plates for temperature actions [VI, XVIV]. Kodysh E.N., Trekin N.N., Nikitin I.K., Samayeva G.D., and Kurmangalieva A.R. considered the issues of substantiation of temperature impact reduction [X, XX]. Scientists such as, for example, Bofang, Z., Makeeva, A., analyzed temperature stresses in massive concrete and reinforced concrete [III, XII]. [III, XII]. Bui K.A., Chen, H., Muneer, K.S., and others studied stresses directly in massive structures of dams and other engineering structures under temperature actions [IV, V, XVI]. Karabanov B.V., Kunin Y.S., and Slesareva A.D. studied the consequences of temperature actions on cast-in-situ reinforced concrete frame [IX, XII, XXI]. Mishchenko N.A. and Slesareva A.D. studied deformations from temperature actions [XIV, XXI].

II. Materials and Methods

The design codes of the Russian Federation, Europe, and the United States all prescribe specific requirements for the design of expansion joints in buildings and other structures. These codes also include instructions on how to calculate the effects of temperature changes on these joints. For example, the Russian code SP 63.13330.2018 "Concrete and Reinforced Concrete Structures: Basic Provisions" states in paragraph 10.2.3 that buildings and structures must be designed with permanent and temporary temperature-shrinkage joints to accommodate temperature changes. According to the codes, the distance between expansion joints is determined based on climatic conditions, the structural features of the building, and the sequence of construction work. In paragraph 1.19 (or 1.22, depending on the reference) of the "Manual for the Design of Concrete and Reinforced Concrete Structures Made of Heavy and Lightweight Concrete Without Prestressing Reinforcement (to SP 52-101-2003)", it is stated that the distance is determined based on both climatic conditions and the order in which the work is done. In this case, if the distance between temperature-shrinkage joints is not greater than the values specified in the manual, it is not necessary to perform calculations. For example, for reinforced concrete cast-in-place frame buildings, the maximum distance between expansion joints should not exceed 50 meters for buildings with heating systems and 40 meters for buildings without heating, according to the manual's table. However, in the latest version of the manual (2005), this information is not included. Instead, it provides general guidelines for constructing expansion joints, similar to those in SP 63.13330.2018, and clause 6.27 of SP 27.13330.2011 "Concrete and Reinforced Concrete Structures Designed for Operation under Conditions of Exposure to Elevated and High Temperatures" also provides a similar table with limits on temperature-shrinkage joint dimensions.

The Eurocodes (EN 1992-1-1) do not provide specific values for the temperature joint spacing requirements in reinforced concrete structures. These requirements are determined in the context of the overall design concept for these structures, taking into account temperature stresses, deformations, and service conditions. For design purposes, it is recommended to refer to national annexes that specify the distances between temperature joints based on the climatic conditions of the country. These distances can vary depending on the specific country, ranging from 10 to 30 meters, depending on the structure and its conditions. It is important to note that the Eurocodes follow a parametric approach to standardization, which aims to standardize final customer properties. The Eurocodes do not specify specific technological methods or solutions, but they do provide unified calculation models and standardized parameters. In contrast, in the United States, joints in concrete and reinforced concrete structures are governed by the ACI 224.3R-95 standard.

Temperature actions on building structures can come from a variety of sources. The most commonly considered types are climatic and technological temperature actions. Climatic temperature actions can be divided into two categories: those related to the closure conditions of structures and those caused by temperature differences across the cross-section of an element, which are related to solar radiation and/or operating conditions.

It is known that temperature (climatic) actions (effects) on structures cause significant stresses in the elements. Free temperature displacement in structures, Δ_t (shortening or elongation of the element), from temperature actions directly depends on the element's length L, temperature difference Δt_w, and linear expansion (shortening) coefficient α, equal to $1 \cdot 10^{-5}$ °C for reinforced concrete, i.e., $\Delta_t = \alpha \cdot L \cdot \Delta t_w$. Therefore, the longer the structure and the larger the temperature difference, the larger the absolute strains ("elongation or shortening") will occur in the element. According to Hooke's law, the normal stresses in the element, σ, depend on the relative deformations of the structure, ε, and the elastic modulus of the material, E., i.e., $\sigma = \varepsilon \cdot E$. And if the element is rigidly fixed on supports (e.g., as in the elements of a frame building), then due to the constraint of absolute deformations in the element, additional stresses inevitably arise in the element from the actions of temperature [XVII, XIII]. Also, when computing structures for temperature actions, not just normal stresses but also tangential stresses, which are always most significant at the supports, should be taken into account.

As can be seen from the above, temperature changes are not as dangerous in heated buildings as they are in unheated or outdoor structures, such as bridges. However, during the construction of heated buildings, when the thermal envelope has not yet been closed, temperature fluctuations can still pose a serious risk to the strength and durability of the structure. This is especially true at the moment when the main load-bearing elements are being installed, whether in heated or unheated structures.

In the present study, the method of calculation for temperature actions according to Russian codes was used.

According to the Russian codes clause 13.1 SP 20.13330 “Loads and impacts” for structures not protected from diurnal and seasonal temperature changes, it is necessary to take into account the change in time Δt of the average temperature and temperature difference ν over the element cross-section, except for cases stipulated by the codes of structural design. For structures protected from diurnal and seasonal temperature changes, temperature climatic actions do not need to be taken into account. That is, for a heated building with a closed thermal circuit, temperature actions are no longer taken into account during operation. However, it should be noted that for the same building during construction, temperature actions can cause sufficiently large and repeated temperature forces and deformations.

Temperature loads occur only in statically undetermined structures. For example, for systems with one superfluous bond, this occurs immediately after the system closure, i.e., when the structure is transformed from statically determinable to statically indeterminable. The system closure is realized at some temperature t_0 - the system closure temperature.

According to the Russian standards, the characteristic values of average temperature changes in the cross-section of the element (at the closure of structures) in warm Δt_w and cold Δt_c seasons, respectively, are determined by the following formulas:

$$\Delta t_w = t_w - t_{0c} \text{ and } \Delta t_c = t_w - t_{0c} \tag{1}$$

The characteristic values of temperatures for warm and cold seasons, t_w and t_c, are given in Table 13.3 of SP 20.13330 “Loads and Impacts”. The values of temperatures t_w and t_c depend on the average daily temperatures of outside air, indoor air temperature, and increments of the average temperatures along the cross-section of the element, temperature difference from daily fluctuations of outdoor air temperature, and temperature difference from solar radiation. Temperature increments in turn depend on the thickness of the structural element, the material of the external surface (solar radiation absorption coefficient), geographical location, and other factors.

The initial temperatures (closing temperatures) in warm and cold seasons, respectively, are determined by the following formulas:

$$t_{0w}=0{,}8\cdot t_{\mathrm{VII}} + 0{,}2t_{\mathrm{I}} \text{ and } t_{0c}=0{,}2\cdot t_{\mathrm{VII}} + 0{,}8t_{\mathrm{I}} \tag{2}$$

Here, t_{I}, t_{VII} - long-term average monthly air temperatures in January and July, taken for the aboveground part of the building according to Table 5.1 of SP 131.13330 “Building Climatology”. For the underground part of the building is accepted that $t_{\mathrm{VII}} = t_{max}(h)$; $t_{\mathrm{I}} = t_{min}(h)$.

When considering uniform heating or cooling, it is important to know at what time of the year the structure will be closed (built). If the structure is built (closed) in winter, Δt_w is chosen as the influence, and vice versa. However, in practice, most structures are complex systems with many redundant connections that are closed at different times. For these structures, it is difficult to accurately determine the temperature at which zero loads would occur. During the construction process, temperature deformations and loads can occur depending on climatic conditions and the order of installation. It is logical to assume that initial loads during construction are lower than temperature loads after the system has been fully closed. If the construction is completed in stages, there will be a distribution of loads over time. Therefore, it is reasonable to use an average temperature value for the construction period as t_0. In the codes, values are given that allow for only one point of closure. Therefore, it can be concluded that, by taking into account the values regulated by the standards, the load is considered in reserve.

Further, we will consider the influence of temperature actions and their consequences on load-bearing structures based on the data obtained during the examination of cast-in-situ reinforced concrete structures of a real facility - a multifunctional residential complex under construction.

The survey focused on cast-in-situ RC load-bearing building structures, including floor slabs, walls, and pillars, with defects in the form of cracks in a multifunctional residential complex with underground parking. The complex consists of three buildings, K1, K2, and K3, located on a single two-level underground section (stylobate). Building K1 is a single-section, 14-story building, while K2 is a two-section, 8-story building, and K3 is a corridor-type, 14-story building (see Figure 3). The functional designation of the complex is residential apartment buildings with underground garages, parking lots, and service facilities for residential development located in built-in, attached, and built-in/attached premises of the apartment buildings. Construction began in 2021, and the survey took place in 2023. Cast-in-place reinforced concrete structures for buildings K1 and K3 were under construction at the time of the survey (from the 14th floor upward). The load-bearing cast-in-place structures of the K2 building and its stylobate had been fully erected at the time of the survey. The underground two-storey portion of the building has a complex shape, with dimensions of 186.51 x 71.18 meters in axes. It is clear from the dimensions of the underground portion of the building that constructing a structure of such size without taking measures to account for temperature-related deformations would have been extremely risky. The design documents did not include any measures to absorb or reduce these temperature-induced deformations.

Fig. 3. General view (photo) of the above-ground buildings of the facility at the time of the survey.

The goals of the survey were to:

- Determine the load-bearing capacity of the building structures of the facility, taking into account its functional purpose, type, and planned loads and impacts during operation, including the presence of cracks in the load-bearing concrete structures.;
- Identify the causes of the cracks in the RC building structures to assess the safety and integrity of the structures.

To achieve the set goals and a comprehensive analysis of the causes of cracking, the following tasks were defined and completed:

1. Analysis of design, operational, execution, and other documentation;
2. Complete visual inspection of the structures of the facility under examination with identification, fixation, and determination of geometric parameters of visually identifiable flaws and damages.
3. Classification and assessment of the influence of the revealed defects on the load-bearing capacity of the inspected structures.
4. Instrumental determination of parameters of defects and damages of building structures of the facility:

- carrying out monitoring of crack opening width during 2 calendar months with the help of plate beacon installation;

- selective non-destructive testing to determine the location of reinforcement by non-destructive electromagnetic induction method using equipment *Proceq Profometer* 650 *AI* (Switzerland) and *POISK* 2.6 (Russia);

- conducting surveys of building structures using a portable *GPR Proceq GP*8000 (Switzerland);

- additional GPR survey methods;

- selective geophysical scanning of foundation structures and subsoil grounds;

- determination of concrete strength by non-destructive methods of control by surface ultrasonic sounding;

- continuous engineering-geodetic surveys to determine deflections in horizontal and deviations in vertical erected reinforced concrete structures, including torsion/eccentricity testing of the facility;

- X-ray phase analysis of selected concrete samples to determine the possible causes of flaws;

5. Creation of a spatial model of the facility in the *LIRA* 10.12 calculation software, verification calculations, and assessment of the bearing capacity of cast-in-situ reinforced concrete structures for subsequent operation, taking into account the design and working documentation provided, as well as the actual data obtained during the inspection;

6. Analysis and identification of causes of defect formation;

7. Elaboration of general conclusions and recommendations based on the results of the survey.

III. Results and Discussion

According to the results of the visual inspection of the facility there were recorded a variety of significant and insignificant flaws and damages of the inspected load-bearing structures were recorded: defects on the concrete surface, cracks of a different character (Figure 4-6):

- In the floor slab of the -2nd underground floor at the level -6,200 (300 mm thickness, concrete grade B40) was found the existence of a widespread network of load cracks with elevations on the lower surfaces of the floor slab, usually in the span, depth of 100-130 mm, width: 0,1 mm and less - 86%; 0,1-0,2 mm 5%; 0,2-0,3 mm 4%; 0,3 mm and more - 5%.

- In the walls of the -2nd underground floor (300 mm thickness, concrete grade B40), the presence of individual vertical or inclined load-induced cracks in the walls (there are individual horizontal cracks), with a width mainly up to 0,1-0,3 mm;

- in the floor slab -1st underground floor at the level -0,900 (400 mm thickness, concrete grade B40) the presence of a widespread network of load cracks with efflorescence on the lower surfaces of the floor slab, usually in the span, a depth of 100-130 mm, with a width 0,1 mm and less - 84%; 0,1-0,2 mm 10%; 0,2-0,3 mm 4%; 0,3 mm and more - 2%.

- In the walls of the 1st underground floor (300 mm thickness, concrete grade B40), the presence of individual vertical or inclined load cracks in the walls (there are individual horizontal cracks), with a width mainly up to 0.1-0.3 mm;

- The presence of a network of hairline cracks in the walls, pylons, and columns of both the underground and above-ground parts of the multifunctional residential complex is observed. These cracks are aligned with the reinforcement framework and indicate underwatering (over-drying) of the concrete surface during the concrete pouring process. Additionally, there are separate load cracks present in columns, pylons, and walls, with widths ranging from 0.1 to 0.3 m.

- In cast-in-situ slabs of the above-ground part of the building (250 mm thickness, concrete grade B35), there are separate force cracks along the lower surface of the slabs. These cracks can be with or without efflorescence and usually occur in the span and have a width of mainly 0.1 mm or less. Additionally, there are some separate cracks with widths of 0.1-0.2 mm and 0.2-0.3 mm.

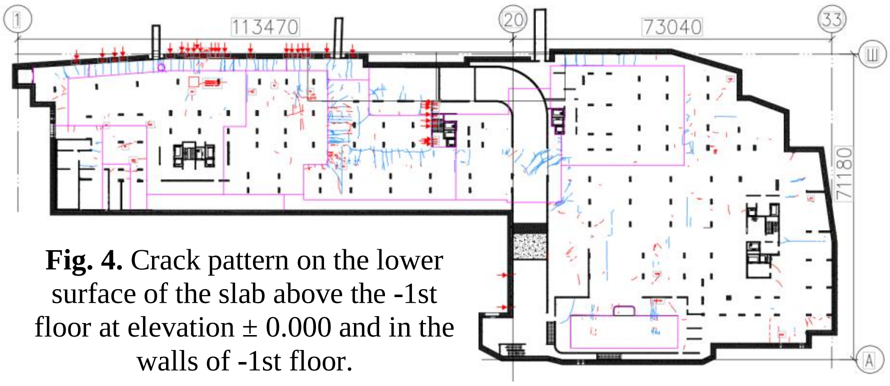

Fig. 4. Crack pattern on the lower surface of the slab above the -1st floor at elevation ± 0.000 and in the walls of -1st floor.

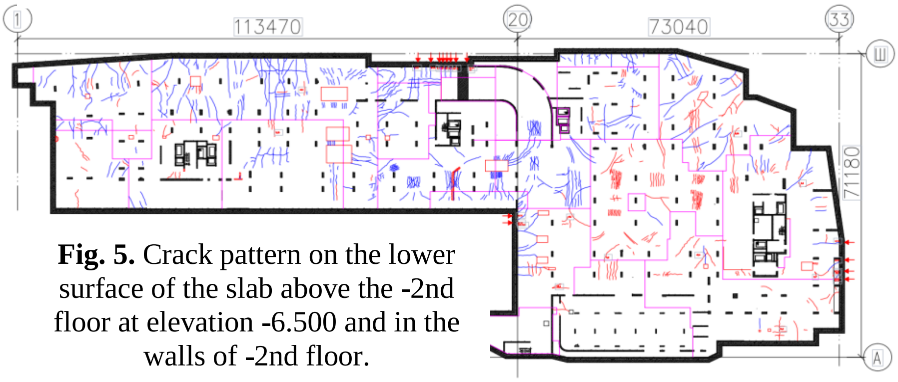

Fig. 5. Crack pattern on the lower surface of the slab above the -2nd floor at elevation -6.500 and in the walls of -2nd floor.

Notes to Figures 4 and 5: Pink indicates cold concrete joints; red indicates cracks of different widths; blue indicates repaired cracks; the red arrow indicates vertical cracks in the walls.

Fig. 6. Cracks detected during the inspection: a - in the floor slab of the 1st floor, b - in the floor slab of the 2nd floor; c - in the exterior walls of the underground - 2nd floor; d - measurement of the crack width (0.2 mm).

The cracks on the bottom surface of the floor slabs are mainly located in the span area, while they occur less frequently at the supports. The maximum crack opening was recorded in the center of the spans. As a rule, the crack opening gradually decreases when approaching the support.

During the inspection, load cracks were detected on the lower surfaces of the floor slabs and can be divided into several groups: (1) cracks in the middle part of the floor slab span; (2) cracks originating at the concrete joints in the floor slab, formed mainly in the middle part of the floor slab span; (3) some cracks had been widened and subsequently repaired with cement-sand composition.

Based on the results of the analysis of the design and construction documentation, the following conclusions have been drawn:

- The building of the multifunctional residential complex with underground parking does not have expansion and contraction joints, according to the design.

- Temperature deformation calculations have not been carried out for the majority of load-bearing structures, or have only been partially carried out for some structures.

According to the results of instrumental inspections, it was established that:

- The reinforcement spacing and the concrete cover thickness in the sampled areas generally match the design specifications;
- According to the results of the random geodesic survey, no tilts or torsion deformations were recorded for buildings 1, 2, and 3;
- The actual concrete grade of the underground portion of the building, for floor slabs, walls, columns, and pylons, corresponds to design grade B40;
- According to the results of the concrete samples for X-ray phase analysis, there were no indications that the cause of the cracks in the structures could be due to the use of low-quality concrete mixes or violations in the concrete pouring process.
- during the monitoring of cracks for two months, it was recorded that some cracks continued to open, while some showed a "closing" trend, i.e. at the time of inspection, there were various alternating deformations (elongation and shortening) in the load-bearing structures of the building, which affected the behavior of the cracks.;
- According to the results of GPR studies, there were no serious anomalies identified in the structure of the RC foundation slab and the soils of the base.

A spatial model of the facility was created in the CAD software, and verification calculations of the main load-bearing structures of the facility were carried out using CAD software *LIRA* 10.12. The calculation of the building structures was carried out for three variants of loading (Figure 3): 1 variant - for the action of service loads; 2 variant - on the action of its own weight and temperature action at the stage of construction; 3 variant - for the action of own weight and temperature action taking into account the construction phase of load-bearing structures.

According to the results of the calculation for variant 1, the actual reinforcement is sufficient for the perception of service loads. According to the results of the calculation for variants 2 and 3, there are significant tensile forces in the elements of floor slabs -1-th and -2-th underground floors, due to which the existing design reinforcement of a set of individual sections of floor slabs -1-th and -2-th underground floors is not sufficient. The effect of humidity on the structures was not taken into account in the temperature calculations.

Figure 7 shows the general FE analytical model of the inspected facility in the *LIRA* 10.12 software. Figures 8-15 show maps with the utilization factor at the stage of construction, taking into account the specified (design) reinforcement.

Fig. 7. General calculation scheme of the inspected facility with LIRA 10.12 software.

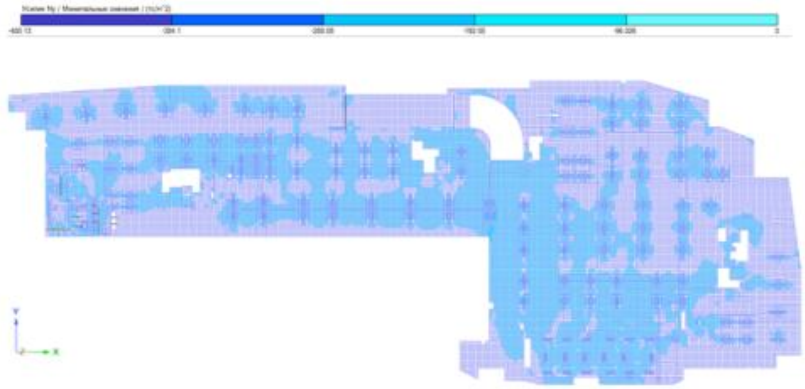

Fig. 8. Maximum values of N_y forces in the floor slab of the -2nd underground floor at -6,200.

Fig. 9. Maximum values of M_x forces in the floor slab of the -2nd underground floor at -6,200.

Fig. 10. Minimum values of M_y forces in the floor slab of the -2nd underground floor at -6,200.

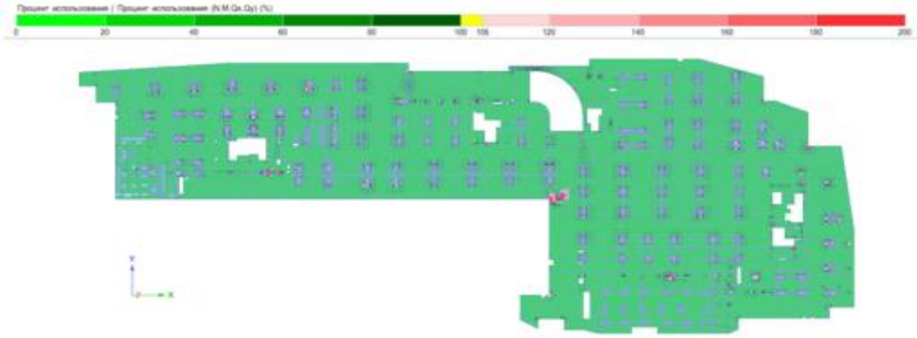

Fig. 11. Utilization factor for -2nd underground floor slab at elevation -6,200 for own weight and temperature actions, considering the specified reinforcement.

Fig. 12. Maximum values of N_y forces in the floor slab -1st underground floor at -0.900.

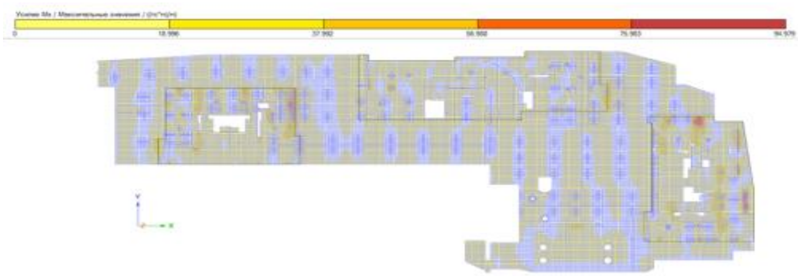

Fig. 13. Maximum values of forces M_x in the floor slab -1st underground floor at the level -0,900.

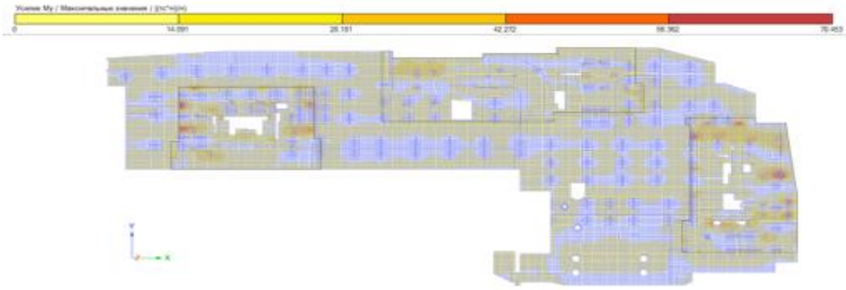

Fig. 14. Maximum values of M_y forces in the floor slab -1st underground floor at -0.900.

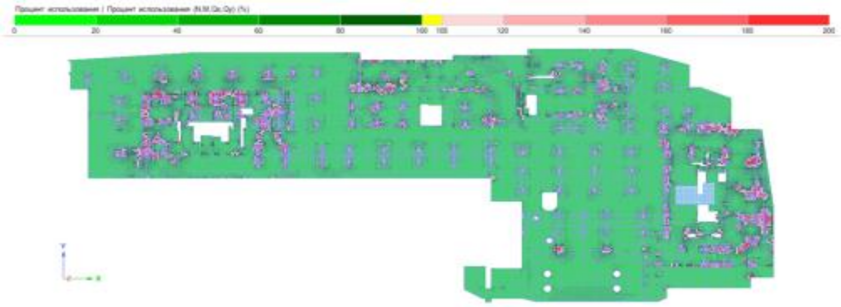

Fig. 15. Utilization factor of the strength of the floor slab -1st underground floor at the level of -0,900 for the action of its own weight and temperature action, taking into account the specified reinforcement.

From the results of the calculation according to Figures 6-13, the following conclusions can be drawn:

- the percentage of exceeding the utilization of the strength of the structures, considering the design reinforcement for the floor slab of the 2nd floor at -6,200 is up to 200% for individual sections (Figure 8-11);

- the percentage of exceeding the utilization of the strength of structures with regard to the design reinforcement for the floor slab -1st floor at the level of -0,900 ranges from 105% to 200% for specific individual sections (Figure 12-15).

Thus, a comprehensive survey of the building was conducted, and various multifactorial survey results were obtained, including the location and nature of the cracks, their width, geodetic survey data, concrete strength measurements, actual reinforcement locations, calculation results, and more. It is worth noting that, according to clause 8.2.15 of SP 63.13330.2018, the maximum safe crack opening width for reinforced concrete structures is 0.3 mm for long-term cracking and 0.4 mm for short-term cracking. According to Eurocode 2, the maximum allowable crack width is w_{max}= 0.4 mm, as shown in Table 7.1 of EN 1992-1-1. Some of the observed cracks exceed these maximum allowable widths. To better understand the specific causes of these different types of cracks and establish causal relationships between the various data points, the survey results must be analyzed.

It is worth noting that the building's frame is rigid and solid. When the plates are cooled or shortened, the supports prevent any free deformation, leading to stretching in the parts farthest from the supports (in the middle of the span). In contrast, during heating or elongation, the process is similar, but the stress patterns change (compression in the span and stretching at the supports). Since there are cracks in the spans, shortening of the plates likely occurred during cooling after hydration or during seasonal cooling. Therefore, the cracks in the span area on the lower surface of the floor slabs can be explained by alternating temperature changes during the construction of the underground part before the completion of the temple's circuit. These temperature changes are superimposed on the deformations in the plate spans, where the maximum tensile stresses occur on the lower surfaces of the plates due to conventional evenly distributed loads. That is why such cracks are less common in supports. As you approach the support, the width of these cracks tends to decrease.

The cause of the cracks on the lower surface of the floor slab has been confirmed by their location within the cells of the slab. These cracks are mainly located in the span of the slab cells, often running crosswise or forming an "envelope" (from the columns to the center of the cell, or from the columns to process openings). They also extend to the cold concrete joints, which are areas of high stress concentration.

Vertical cracks in exterior walls can also be explained by temperature-related deformations, which are more pronounced in the longest walls because the linear elongation or shortening, according to Hooke's law, directly depends on the length of the wall. It is also important to consider the influence of factors such as shrinkage contraction of the walls during both temperature changes and drying processes. The walls experience both temperature shrinkage and drying shrinkage, which is constrained by the rigidness of the foundation slab underneath, the rigidness of floor slabs resting on the walls, and the rigidness of adjacent walls. The strength provided by the solid foundation slab at the bottom and floor slabs on top prevents free compression and causes tensile stresses that exceed the tensile strength of the concrete. All this is confirmed by the fact that vertical cracks in walls are observed in places where the wall's cross-section changes or when the orientation of the walls is changed, that is, in areas where the greatest stress is concentrated. In other words, there is a "locking" effect of deformations in the corners of buildings, where stress concentration is always higher. Additionally, the potential effect of variations in wall thickness, hydration, and cooling leading to uneven shrinkage and self-regulating stresses further increases the likelihood of cracking.

A network of hairline cracks in the floor slabs, walls, pylons, and columns of the underground and aboveground parts of a multifunctional residential complex indicates under-wetting or over-drying of the concrete surface during the concreting process.

The presence of local cracks in the monolithic floors, columns, pylons, and walls of the aboveground building, with or without efflorescence, usually with a width ranging from 0.1 to 0.3 millimeters, suggests their complex nature, likely due to local stresses caused by overdrying during concreting. These stresses manifested themselves in areas of high tensile stress.

It is worth noting that there are some discrepancies between the results of the verification calculations for loads and impacts during the construction phase and the actual situation of cracks in the load-bearing structures of the first and second floors.

The real crack distribution map shows that most cracks are recorded primarily on the lower surfaces in the middle of the floor slabs on the first and second floors, whereas according to the results of verification calculations, there is a lack of design reinforcement primarily in the support areas of the floor slabs for these floors.

According to the verification calculations, areas with a lack of calculated reinforcement are significantly larger on the first floor than on the second, while the actual crack formation map indicates that the second floor has a much wider distribution of cracks than the first.

The above inconsistencies need to be clarified. The more intense cracking on the second floor may be due to possible differences in the temperature between floors. In other words, the calculated temperature difference may differ from the actual one, both in terms of the temperature of individual structural sections and the duration of exposure to particular climatic conditions and work processes (heating of concrete, different times of excavation for individual structural sections, etc.).

The force maps (Figs. 8-10 and 13-14) illustrate the maximum tensile stresses and bending moments in the middle of the plate spans under the influence of temperature, which corresponds to the locations where cracks are likely to occur. However, the actual locations of cracks may differ from the map of the areas with a lack of reinforcement, according to the calculations. This is because, on the one hand, temperature fluctuations can vary from actual climate conditions. On the other hand, when there is a shortage of longitudinal reinforcement in certain areas, cracks can occur in flat floor slabs in both the span lower zone and the support zone at the same time and evenly, as the structure tends to maintain a certain level of equilibrium, with stresses being redistributed over the entire floor slab. It is also worth noting that cracks are more likely to occur in the most stressed areas (e.g., spans and supports) or weaker areas (e.g., cold joints or technological openings). During the survey, it was not possible to determine the exact location of cracks in the supports of the slabs, as a cement-sand screed had been applied to the surface of the floor slabs on the second and first floors at the time of the inspection.

The temperature gradient for floor slabs is most pronounced for thicker slabs of 0.3 meters. The observed cracks on the bottom surface in the middle span are consistent with the combined effects of limited overall thermal contraction and potential temperature gradients through the slab thickness, where the top surface cools faster than the bottom during the curing of concrete, which can lead to intense stress at the bottom in areas of maximum bending moments. Therefore, for thicker floor slabs on the first floor (400 millimeters), this factor has a greater impact, and crack opening is more pronounced than for thinner slabs on the second floor (300 millimeters).

The specific crack patterns observed – predominantly mid-span cracks in floor slabs and vertical cracks in walls – are directly attributable to the combined effects of: (1) Restrained overall thermal contraction and expansion; (2) Restrained drying shrinkage; (3) Bending induced by potential thermal gradients through the thickness of slabs and walls; (4) Stress concentrations at cold joints; (5) High boundary stiffness provided by interconnected structural elements and foundations. The variation in crack severity between walls is explained by differences in orientation, length, degree of restraint (highest at corners and between stiff elements), and potentially local variations in curing conditions. Ignoring factors such as tightness, temperature gradient, and shrinkage in calculations and design has led to a local shortage of reinforcement, specifically in the areas where tensile stresses are predicted to be concentrated by the model.

The calculated tensile strength of heavy concrete of grade B40, according to Table 6.7 of SP 63.13330.2018, is $R_{bt,\, ser}$ = 2.1 MPa. For comparison, the average tensile strength of concrete C32/40 (which is close to B40), as indicated by Eurocode 2 (EN 1992-1-1), is $f_{ctm} \approx 3.0$ MPa. The maximum calculated stresses in the floor slab on the 1st floor, N_y = 4.96 MPa, exceed $R_{bt,\, ser}$ by 2.36 times, which explains the intense cracking observed in the areas under consideration.

The computational model indicates the occurrence of significant tensile forces along the length of the plates and walls (N_y, N_x), which fully corresponds with the observed crack pattern. Transverse cracks in the plate spans (Figs. 4a and 4b), as well as vertical cracks in the walls (Fig. 4c), predominate. A comparison of the calculated stresses and estimated crack widths obtained from the *LIRA* model with the requirements and limitations of current regulatory documents (SP 63.13330.2018 and EN 1992-1-1) confirms the adequacy of the model used and its ability to predict stress levels leading to crack formation in a real structure.

The simulation results in *LIRA* 10.12 were validated by comparing the calculated tensile stresses and crack widths with the maximum permissible values in the standards SP 63.13330.2018 and EN 1992-1-1. Additionally, a clear spatial and qualitative correlation was demonstrated between the zones of high stresses and deformations in the model and the actual cracks observed. This validation confirmed the adequacy of the model for assessing temperature effects during the construction process.

Summing up the analysis of possible causes of crack formation, we can conclude that, despite some inconsistencies in the distribution of cracks on the maps, the formation of cracks in the load-bearing structures on the second and first floors is confirmed by calculations based on their own weight and temperature effects. However, ignoring factors such as tightness, gradients, and shrinkage during calculations and design has led to a local lack of reinforcement precisely in areas where tensile stresses are predicted to be concentrated by the model.
It is also worth noting that during the present survey, other possible causes of cracking were excluded, such as uneven foundation settlement, slopes (slopes), deviations from the vertical, other deformations, twisting of the building frame, insufficient concrete strength, inconsistency of the actual reinforcement with the design, inconsistency of the chemical composition of the concrete mix with the declared one, use of substandard concrete mixes, and violations in concrete pouring technology, etc.

Thus, the results of the calculation for the action of its own weight and temperature action at the stage of construction showed that, taking into account the design thicknesses and cross-sections of load-bearing elements included in the design scheme, the design calculated reinforcement is not sufficient to ensure the load-bearing capacity of the building.

It is worth noting that, according to the calculation results, to absorb all tensile forces from temperature actions in the bearing cast-in-situ structures of walls and floors of the stylobate, it would be necessary to lay up to 2 times more design reinforcement, which would be economically unreasonable.

V. Conclusions

Thus, it can be concluded that cracks in the load-bearing structures of the floor slabs on the 2nd and 1st floors were caused by insufficient design reinforcement to withstand all types of impact, including temperature and shrinkage, during the construction stage. The main reason for this is the project's decision to omit expansion joints in such large-scale underground load-bearing structures (dimensions of 186.51 x 71.18 meters). Local cracks in above-ground load-bearing structures were caused by a violation of construction and installation technology during the building's construction (local drying of concrete). To prevent further damage, cracks in the facility's load-bearing structures require repair before commissioning.

Importantly, the construction phase, particularly the early-age period of concrete (first 3-7 days), is identified as the most critical window for crack initiation due to high heat of hydration, rapid evaporation, and the development of significant thermal gradients and restrained shrinkage stresses before the concrete gains sufficient tensile strength. Therefore, mitigating early-age cracking requires a holistic approach combining improved design practices (temperature calculations, joints) with strict adherence to specific field measures and construction technologies focused on thermal and shrinkage control.

Based on a comprehensive investigation combining field examination, instrumental monitoring, and validated numerical modeling (LIRA 10.12), the following conclusions are drawn:

1. The extensive network of cracks observed in the load-bearing floor slabs (-2nd and -1st underground levels) and external walls stems from insufficient design reinforcement to resist combined stresses, including significant temperature-induced forces and restrained shrinkage during construction. Specific crack patterns (mid-span cracks in slabs, vertical cracks in walls) are directly attributable to:

- Restraint of thermal/shrinkage deformations by interconnected structural elements and foundations.
- Thermal gradients through element thicknesses exacerbate bending stresses.
- Stress concentrations at construction cold joints.

2. The fundamental cause is the absence of expansion joints in the design of the large-scale (186.51×71.18 m) underground structure. This omission prevented the accommodation of inherent volumetric changes, leading to critical stress buildup exceeding the capacity of the as-designed reinforcement (utilization factors reached 105-200% in critical sections).

3. Numerical simulations, validated against field crack patterns, dynamic crack monitoring data, and code-specified tensile stress limits (SP 63.13330.2018 $R_{bt,ser}$ = 2.1 MPa, EN 1992-1-1 $f_{ctm} \approx 3.0$ MPa), confirmed stresses up to 3.5 MPa – significantly exceeding concrete tensile capacity and correlating with observed crack widths (up to 0.3 mm).

4. Following structural closure and enclosure of the thermal envelope, the complex is structurally sound for intended service loads. However, the identified cracks require remediation before occupancy.

5. All documented cracks in the underground slabs, walls, and localized above-ground elements require injection repair before commissioning to ensure durability, watertightness (where applicable), and restore serviceability.

Based on the findings of this study and to address the critical early-age period, the following specific field measures are strongly recommended for the construction of cast-in-situ reinforced concrete structures, especially large-scale or restrained elements like slabs and walls in underground floors:

1. Temperature Sensors & Monitoring. Implement a real-time temperature monitoring system within critical structural elements (e.g., center and near surfaces of thick long walls ≥0.3 m, mid-span and support zones of thick slabs ≥0.3 m) using embedded thermocouples or data loggers. Track the evolution of the heat of hydration and identify core-to-surface temperature gradients (ΔT). The target maximum core temperature should ideally be kept below 70°C, and the core-to-surface gradient (ΔT) should be strictly controlled to less than 20°C (based on common guidelines like ACI 207.2R, CIRIA C660) to minimize thermal cracking risk. Monitoring allows for timely intervention (e.g., adjusting formwork insulation).

2. Staged Concreting / Placement Sequencing. For long elements (walls longer than 15 m, slabs larger than 300 m²), it is necessary to use staged concreting. The element should be divided into smaller, manageable segments using temporary formwork partitions. Sequential concreting will ensure partial heat dissipation and stress relaxation in previously constructed sections before the adjacent sections are poured. Joints must be arranged per the requirements of the codes. Reducing the peak temperature rise and the total volume of concrete simultaneously generating heat will reduce the restraining forces and temperature gradients. These measures will allow the heat to dissipate gradually.

3. Control of Heat Generation. Special concrete mixtures with lower heat of hydration can be used. For example, special Portland cement or special cement additives that significantly reduce peak temperatures and slow down the rate of strength gain, improving crack resistance in the early stages. Hydration stabilizing admixtures or shrinkage reducing admixtures (SRA) can be used. SRA can reduce drying shrinkage deformation by 20-50%.

4. Insulation Blankets & Curing. Immediately after placement, exposed concrete surfaces (especially slab tops and exposed wall surfaces) should be covered with insulation blankets or mats. The insulation should be maintained until the core temperature has gradually cooled (target cooling rate < 1-2 °C per hour). After the initial setting of the concrete mix, continuous wet curing (e.g., continuous water

spray) should be maintained for at least 7 days or until 70% of the specified strength is achieved. This is critical to prevent plastic shrinkage cracks in the first few hours/days and to mitigate long-term drying shrinkage.

5. Timing of Formwork Removal. Premature removal of vertical formwork (walls, columns) should be avoided. Formwork should be kept in place for at least 3-7 days, using temperature monitoring data to ensure that the concrete has cooled sufficiently and gained the required strength. Rapid formwork removal exposes young concrete to rapid cooling and drying, which increases the risk of cracking.

The implementation of these specific field measures in combination with basic design methods, including mandatory thermal analysis and the inclusion of expansion joints in the design, provides a comprehensive strategy for reducing the risk of early thermal and shrinkage cracking in monolithic reinforced concrete structures, especially in large-scale or highly limited deformations, such as the surveyed multifunctional complex. Ignoring the design calculations for thermal effects or monitoring concreting at critical stages of construction will inevitably lead to a decrease in serviceability and durability.

The recommendations for installing temperature joints or performing special calculations, as indicated in this work, are supported by the findings of studies [IX, XXII, XXIII]. These studies indicate that the absence of these measures can lead to massive cracking.

Conflict of Interest:

There was no relevant conflict of interest regarding this paper.

References

I. Aleksandrovsky, S. V. *Calculation of Concrete and Reinforced Concrete Structures for Temperature and Humidity Effects (Taking into Account Creep).* Moscow: Stroyizdat, 1966.

II. Arangi, S. R., and R. K. Jain. *"Review Paper on Pavement Temperature Prediction Model for Indian Climatic Condition." International Journal of Innovative Research in Advanced Engineering (IJIRAE)*, vol. 8, no. 2, 2014, pp. 1–9.

III. Bofang, Z. *Thermal Stresses and Temperature Control of Mass Concrete.* United States of America, 2014, p. 518. 10.1016/C2012-0-06038-3.

IV. Bui, K. A., P. Sancharoen, G. Tanapornraweekit, S. Tangtermsirikul, and P. Nanakorn. *"An Evaluation of Thermal Effects on Behavior of a Concrete Arch Dam." Songklanakarin Journal of Science and Technology*, vol. 41, no. 5, 2019.

V. Chen, H., and Z. Liu. *"Temperature Control and Thermal-Induced Stress Field Analysis of GongGuoQiao RCC Dam." Journal of Thermal Analysis and Calorimetry*, vol. 135, no. 3, 2018, p. 11. 10.1007/s10973-018-7450-1.

VI. Chepurnenko, A. S. *Improving the Methods for Calculating Plates and Shells for Force and Temperature Effects during Nonlinear Creep: Specialty 05.23.17 "Structural Mechanics": Dissertation for the Degree of Doctor of Technical Sciences.*, 2021, p. 349.

VII. Do, T. M. D., and T. Q. K. Lam. *"Solutions to Improve the Quality of Mass Concrete Construction in Climate Conditions of Southern Vietnam." International Journal of Innovative Technology and Exploring Engineering (IJITEE)*, vol. 8, no. 6C2, 2019, pp. 188–192.

VIII. Gordeev, V. N., A. I. Lantukh-Lyaschenko, V. A. Pashinsky, A. V. Perelmuter, and S. F. Pichugin. *Loads and Impacts on Buildings and Structures*. Moscow: ASV, 2007, p. 476.

IX. Karabanov, B. V. *"Features of Calculation of Monolithic Reinforced Concrete Buildings for Temperature Effects." Concrete and Reinforced Concrete*, no. 1, 2010.

X. Kodysh, E. N., N. N. Trekin, and I. K. Nikitin. *Design of Multi-Storey Buildings with a Reinforced Concrete Frame.*, 2009, p. 347.

XI. Korsun, V. I., and Khon K. *"Strains and Strength of Reinforced Concrete Beams Manufacturing by High-Strength Concrete for Non-Coincident Planes of Temperature Gradient and Loading." Construction of Unique Buildings and Structures*, no. 109, 2023, Article no. 10914. 10.4123/CUBS.109.14.

XII. Kunin, Yu. S., V. I. Kotov, and L. Kh. Safina. *"Reduction of Operational Qualities of a Building as a Result of Design Errors. Part 1." Scientific Review*, no. 7, 2017, pp. 36–40.

XIII. Makeeva, A., A. Amelina, K. Semenov, and B. Yuriy. *"Temperature Action in Analysis of Thermal-Stressed State of Massive Concrete and Reinforced Concrete Structures." MATEC Web of Conferences*, vol. 245, 2018, Article 03016. 10.1051/matecconf/201824503016.

XIV. Mishchenko, N. A. *"Methodology for Determining Temperature Deformation of Building Structures." Engineering Geodesy*, no. 32, 1988, pp. 69–71.

XV. Mkrtychev, O. V., and D. S. Sidorov. *"Calculation of Reinforced Concrete Building for Temperature Effects." Vestnik MGSU*, no. 5, 2012, pp. 50–55.

XVI. Muneer, K. S., K. R. Muhammad, H. B. Mohammed, and A. T. Lutf. *"Cracking in Concrete Watertank Due to Restrained Shrinkage and Heat of Hydration: Field Investigations and 3D Finite Element Simulation." Journal of Performance of Constructed Facilities*, vol. 34, no. 1, 2020, p. 12. 10.1061/(ASCE)CF.1943-5509.0001356.

XVII. Nguyen, T. C., K. A. Bui, and Q. L. Hoang. *"Thermal Cracks in Concrete Structure – The Basic Issues to Be Understood." Structural Health Monitoring and Engineering Structures. Lecture Notes in Civil Engineering*, vol. 148, June 2021. 10.1007/978-981-16-0945-9_19.

XVIII. Plotnikov, A. A. *"Taking into Account Temperature Effects in the Design of Load-Bearing Structures." Housing Construction*, no. 11, 2021, pp. 21–26. 10.31659/0044-4472-2021-11-21-26.

XIX. Sabirov, R. A. *"On the Calculation of Plates under Plane Stress Conditions for Temperature Effects Using the Variational-Difference Method in Stress Functions." Vestnik of the Siberian State Aerospace University Named after Academician M.F. Reshetnev*, no. 2 (54), 2014, pp. 67–72.

XX. Samaeva, G. D., and A. R. Kurmangaliyeva. *"Justification of Expansion Joints of a Multi-Story High-Rise Residential Complex along Bakinskaya Street." Innovative Development of Regions: The Potential of Science and Modern Education, Proceedings of the VI National Scientific and Practical Conference*, Astrakhan, 8–9 Feb. 2023, no. 6, pp. 59–64.

XXI. Shelkova, E. O., A. I. Korosteleva, and N. V. Sheshenev. *"Analysis of Design Measures in the Device of Expansion Joints." New Technologies in the Educational Process and Production: Proceedings of the XVII International Scientific and Technical Conference*, Ryazan, 17–19 Apr. 2019, pp. 203–206.

XXII. Slesareva, A. D. *"Displacements of a Residential Building Structure under the Influence of Temperature Effects Arising from Solar Radiation." Student Bulletin*, vol. 15, no. 7 (301), 2024, pp. 29–32.

XXIII. Snegirev, A. I., and A. I. Alkhimenko. *"Effect of Short-Circuit Temperature during Construction on Stresses in Load-Bearing Structures." Engineering and Construction Journal*, no. 2, 2008.

XXIV. Vladimir, I. K., Valeriy, I. M., Ashot, G. T., and Anatoly, V. A. *"Nonlinear Deformation Model for Analysis of Temperature Effects on Reinforced Concrete Beam Elements." Buildings*, vol. 13, 2023, pp. 1–16. 10.3390/buildings13112734.

XXV. Voldřich, F. *Expansion Joints in Structures of Above-Ground Buildings.* Translated from Czech by T. M. Vankevich, Moscow: Stroyizdat, 1978, p. 224.