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Engineering structures stability against progressive collapse

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Abstract. The calculating stability of an engineering structure against progressive collapse is considered in this article. The calculations results are analyzed. A dangerous emergency scenario was developed and measures were developed to prevent the progressive building collapse in the structures local destruction in accordance with the regulatory documents requirements for such an object. The most dangerous design schemes are considered to assess the structure stability against progressive collapse. A scenario for calculating stability against progressive collapse in the case of local failure has been selected, and a consistent action algorithm has been obtained that makes it possible to perform a calculation for resistance to progressive collapse., The kinematic method of the limit equilibrium theory for structures with elastic-plastic materials properties is used to calculate the resistance to progressive collapse. It corresponds to the turning the system into a plastic kinematic chain, the links movement is carried out due to the plastic flow of the system's links. Two types of undamaged structures were identified to assess the building resistance to progressive collapse: neighbouring beams and columns, in which local destruction impact does not cause a qualitative change in the stress state, but leads to an increase in stresses and forces (neighboring beams, columns); pavement slabs resting on a beam and welded through embedded parts to a beam that has lost its original support, and located above a local fracture, the stress state changes. The conclusion is made about the requirements that should be met in order to conclude the calculation justification so it can be said that resistance to progressive collapse is ensured. It has been established that the calculations performed confirm the object stability - the drainage chamber to progressive collapse in the local destruction as an emergency situations result.

Keywords: progressive collapse, stability, kinematic method of the limit equilibrium theory

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1 Introduction

Nowadays, as a result of the increased buildings and structures accidents, which are caused by such reasons as design errors, construction technology violations, non-compliance with operating rules, and so on, the problem of ensuring the buildings and structures stability to progressive collapse has been given special attention both in the Russian Federation and abroad. According to Russian researchers, the damage from the object collapse is estimated at no less than 700%. The article considers the calculating the stability issue of an engineering structure against progressive collapse, and also analyzes the performed calculations results.

First of all, loads were collected for subsequent calculations (Table 1). The existing loads were determined in accordance with SP 20.13330.2016 «Loads and Impacts».

Table 1. Self-weight load of coating structures per 1m^2

No	Name	Density (kg/m^3)	Layer thickness (mm)	Safety factor γ_f	Load (kg/m^2)	
					Estimated	Normative
1	Technoelast 2 layers	3	8.2	1.3	3.9	3
2	Cement strainer	1800	50	1.1	99	90
3	Mineral wool boards	135	120	1.3	21.1	16.2
4	Priming with bitumen	1	-	1.3	1.3	1
5	Coating slabs			1.1	181.5	165
Total:					306.8	275.2

Temporary loads determination. First, the snow load. The normative snow load on the horizontal projection was calculated using the formula 1:

$$S_0 = c_e \cdot c_t \cdot \mu \cdot S_g \tag{1}$$

where c_e –coefficient taking into account the snow removal from buildings under the wind influence or other factors;

c_t–thermal coefficient;

μ –the transition coefficient from the snow cover weight on earth to the snow load on the cover;

S_g - the normative snow cover weight per 1 m^2 of the horizontal earth surface. S_g = 150 kg/m^2 - for III snow region.

The coefficients c_e, c_t, μ for the roof:

1. The coefficient μ for buildings with a flat roof is 1.
2. The coefficient c_e is determined according to the formula 2:

$$c_e = (1.2 - 0.4 \cdot \sqrt{k}) \cdot (0.8 + 0.002 \cdot l_c) \tag{2}$$

k is taken according to table 11.2 for terrain types A or B;

$l = 2 \cdot b - \frac{b^2}{l}$ - characteristic slab size, taken no more than 100m;

b - the smallest size of the slab in the plan;

l - the largest size of the slab in the plan.

The coefficient k is determined by interpolation for a building with a height of 7.95 m. k =0.59;

$$l = 2 \cdot 12.65 - \frac{12.65^2}{36.8} = 25.3 - 4.3 = 21 \text{ m};$$

$$c_e = (1.2 - 0.4 \cdot \sqrt{0.59}) \cdot (0.8 + 0.002 \cdot 21) = 0.75$$

3. Coefficient c_t definition. c_t =1 [1-2].

$$S_0 = 0.75 \cdot 1 \cdot 1 \cdot 150 = 112.5 \text{ kg/m}^2$$

The design value of the snow load is determined by the formula 3:

$$S = S_0 \cdot \gamma_f \tag{3}$$

$$S = 112.5 \cdot 1.4 = 157.5 \text{ kg/m}^2$$

Snow load near the pavement parapet at $h = 0.5 < \frac{S_0}{2} = \frac{1.12}{2} = 0.56$.

The loads on the beam along the 2/1 axis have been collected. The length is l=12 m (beam dead weight is 6 t), taking into account the special combination of loads in case of emergency failure. Self-weight load of roofing slabs and roofing pie:

$$q = 275.2 \text{ kg/m}^2 \cdot 6 \text{ m} = 1651.2 \text{ kg/m} = 16.5 \text{ kN/m}$$

The snow load on the beam in an emergency is 472.5 kg/m, taking into account the coefficient 0.5. The total load on the beam (roof dead weight + snow load) is 21.2 kN/m.

Loads on the column:

$$Q = 21.2 \cdot 6 + \frac{60}{2} = 154.2 \text{ kN}$$

2 Materials and methods

A dangerous emergency scenario was developed for the structure in accordance with the regulatory documents requirements, and measures were developed to prevent the building progressive collapse in case of local structures destruction. The most dangerous design schemes are considered to assess the structure stability against progressive collapse. The calculation scenario for stability against progressive collapse in the case of local destruction, including the column destruction in the extreme row along the «2/1» axis, was chosen [3-6]. The columns destruction scheme along the axis «2/1» is shown in Figure 1.

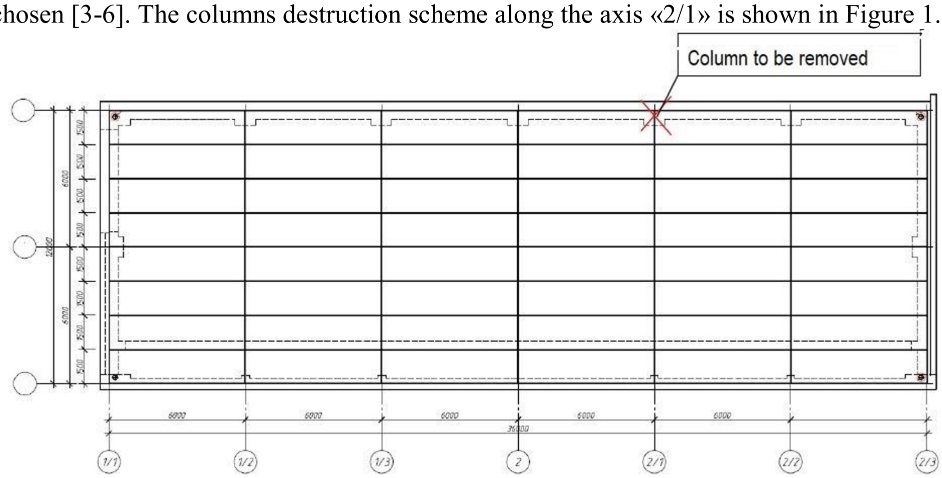

Fig. 1. Calculation scheme for the columns destruction along the axis «2/1»

Two types of undamaged structures were identified to assess the building resistance to progressive collapse:

1) in the first type elements - neighboring beams and columns - the local destruction impact does not cause a qualitative change in the stress state, but leads to an increase in stresses and forces (neighboring beams, columns). These structures overloading is not more than twice.

2) in the second type elements - coating slabs supported on a beam and welded through embedded parts to a beam that has lost its original support and located above a local fracture - the stress state changes in the state under consideration [7].

The main calculation task is to check the pavement slabs stability supported by beams located above the local destruction and lost support as a result of the column local destruction (removal). Stability depends both on the elements strength themselves and on the links strength. Sustainability is ensured through the rational links design between prefabricated elements., The design solutions include special plastic elements made of reinforcing steel (class A500C) to ensure the joints plasticity of prefabricated reinforced concrete slabs. A stretched linear connection between prefabricated slabs is a sequentially connected elements chain - an embedded part anchor, an embedded part, the link-up itself, an embedded part of another slab, its anchor. The work provides for the individual coating slabs connection into a single disk with special metal links [8-10].

The design characteristics of the materials resistance are presented in Table 2.

Table 2. Design characteristics of the materials resistance

Material	Class	Compression	Tension
Concrete	B20	$R_{b,n}$, $R_{b,ser}$ = 150 kg/cm^2	$R_{bt,n}$, $R_{bt,ser}$ =13,5 kg/cm^2
Reinforcement	A500C	R_{sn} =5000 kg/cm^2. E = 200000 MPa	

The kinematic method of the limit equilibrium theory with elastic-plastic materials properties is used to calculate the resistance to progressive collapse. It corresponds to the turning the system option into a plastic kinematic line, the points movement is carried out due to the plastic flow of the system's links.

In the emergency failure of one of the columns and a possible slab collapse, it is recommended that special floor slabs links provide effective resistance to the progressive slabs collapse at large deflections, as hanging system elements. The requirements that the links and slabs that form the system should follow from the calculation according to the deformation scheme (Figure 2): the series-connected elements (link-up - slab - link-up - slab – link-up) should include a very plastic link - in our case: links from reinforcing steel (class A500C), which would provide a total elongation of the chain of ε ≈ 5% (in this case, any cracks are allowed in the slabs) [11-15].

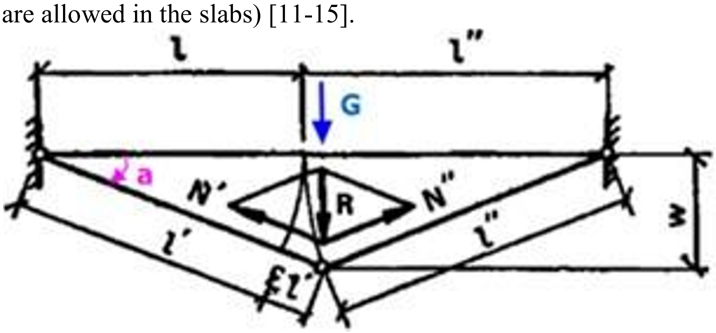

Fig. 2. Deformation scheme

3 Results and Discussion

Conditions 4 and 5 must be taken into account to fulfill this condition:

$$N \cdot \sqrt{2 \cdot \varepsilon \cdot (1 + \frac{l_{min}}{l_{max}})} \geq G \tag{4}$$

$$\omega \leq l_{min} \cdot \sqrt{\frac{2 \cdot \varepsilon}{1 + \frac{l_{min}}{l_{max}}}} \tag{5}$$

where G –is the line load falling on the beam that has lost its support (with supporting column local destruction). In this case:

$$\mathrm{G_n} = 21.2 \frac{\mathrm{kN}}{\mathrm{m}} + \frac{60\ \mathrm{kN}}{12\ \mathrm{m}} = 26.2\ \frac{\mathrm{kN}}{\mathrm{m}}$$

where 2.12 m–the maximum load on the beam (for calculating the link-up).

Linear bearing capacity of the hanging chain weakening link is:

$$N \geq \frac{26.2}{\sqrt{2 \cdot \frac{5 \cdot 2}{10}}} = 58.6\ \frac{\text{kN}}{\text{m}}$$

The equilibrium deflection is achieved when ω=1.34 m. Required area of link reinforcement per slab at R_c = 5000 kg/cm^2:

$$A_{s,n.} = \frac{5.86 \cdot 1.5}{5} = 1.76\ \text{cm}^2$$

Four links (Ø16 A500C) with an area A_s =8 cm^2 are installed for each slab, which is more than $A_{s,n.}$ = 1.76 cm^2. The line load on neighboring beams (first type elements) increases to 34.3 kN/m, q = 21.2 kN/m ·1.5 m + $\frac{60\ kN}{2\cdot 12\ \text{m}}$ = 34.3 kN/m, which is less than the allowable load equal to 42 kN/m - permissible load on the roof beam frame. The load on adjacent columns (first type elements) in an emergency N = 206 + $\frac{60}{2}$ = 236 kN, which is less than the column bearing capacity.

The building stability against progressive collapse during local column destruction along the «1/2» (or «2/2») axis (at a distance of ≈ 6 m from abutting end) is also tested. It is planned to install additional horizontal tension links connecting the outer wall panels (the abutting end - axes «1/1» (or «2/3») with the embedded slabs parts of the extreme row (at the abutting end). In this case, the outer wall panels resist progressive collapse with half-timbered columns along the axis «1/1» (or «2/3»); tensile links connecting the outer wall panels through the channel holder with the slabs and special links between the coating slabs [16-20].

4 Conclusions

The following conclusions can be drawn based on the performed calculations:

1. The forces are redistributed through reinforcing lines to neighboring beams and columns when the extreme row column removal is modeled.
2. Forces in neighboring columns, beams (first type elements) are less than their calculated bearing capacities, the condition F ≤ S (where F is the force from the calculation, S is the actual bearing capacity) is observed.
3. The performed calculations confirm the building resistance to progressive collapse in local destruction (column removal) as a result of emergency situations.
4. The building stability is ensured after the special reinforcing links installation (Ø16 A500C), the bearing capacity is greater than that required by the calculation.
5. The collapse impossibility of suspended roof structures - prefabricated slabs based on a beam that has lost its support and located above the destroyed column («1/2» (or «2/2») axes) is ensured due to the rational design of links between prefabricated elements:

• special links made of reinforcing steel (Ø16 A500C) between prefabricated slabs;

• additional special links made of reinforcing steel (Ø16 A500C), connecting the outer wall panels (along the axes «1/1» (or «2/3») with embedded parts of prefabricated roof slabs [21-23].

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