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STRESS STATE OF THE SERIAL RESIDENTIAL BUILDING IN SAMARKAND UNDER SEISMIC ACTION BEFORE AND AFTER REINFORCEMENT

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АНОТАЦІЯ: В статті розглянутий напружений стан цегляної серійної житлової будівлі в м. Самарканд Республіки Узбекистан при сейсмічних впливах до та після її підсилення з врахуванням нових норм.

АННОТАЦИЯ: В статье рассмотрено напряженное состояние кирпичного серийного жилого здания в г. Самарканд Республики Узбекистан при сейсмических воздействиях до и после его усиления с учетом новых норм.

ABSTRACT: The article deals with the stress state of a masonry residential 310 series building in Samarkand, Uzbekistan under seismic action before and after its reinforcement in accordance with the new standards.

KEY WORDS: seismicity, stress, masonry, reinforcement.

Wide spread occurrence of old residential buildings since the Soviet Union and seismic areas at the same time in Uzbekistan makes it relevant to study the issue of seismic resistance of such buildings.

The apparent discrepancy between the old residential buildings and the new anti-seismic construction standards creates the problem and necessitates studying the issue and topics of an effective improvement of such buildings earthquake resistance.

Analysis of the available literature sources showed that until recently in

Republic of Uzbekistan no complex studies of seismic resistance of old residential brick buildings taking into account the new standards, including the effectiveness of its enhancement, had not been carried out.

The aim of the study described in the article is to examine the stress state of a representative building under seismic action before and after its reinforcement in terms of seismic resistance.

As the main experimental building there was selected a residential 310 series building being one of the most characteristic and problematic and used until quite recently. Data concerning its constructive and architectural-planning design, strength of the basic materials of construction etc. were obtained as a result of the full-scale survey and literature review. Thus, we got all necessary characteristics for modeling and calculation of the considered series building. Since the major defect revealed in the surveyed building structures and construction materials was a low strength of mortar (M25 instead of M50) it was decided to take into account this particular defect in calculations through appropriate corrections of masonry rigidity and load-bearing capacity.

For the initial assembly of the building model there was used the software of PC "MONOMAKH-SAPR" which significantly facilitated this procedure as compared to that of PC "LIRA-SAPR". Constant loads and temporary ones specified by the norms were applied [1]. The model made by means of PC "MONOMAKH-SAPR" was exported to PC "LIRA-SAPR" taking into account the basic finite element size of 0.4 m. Final modification of the model was completed using PC "LIRA-SAPR". Rigidity of the building base was considered. General view of the modified model is shown in fig. 1.

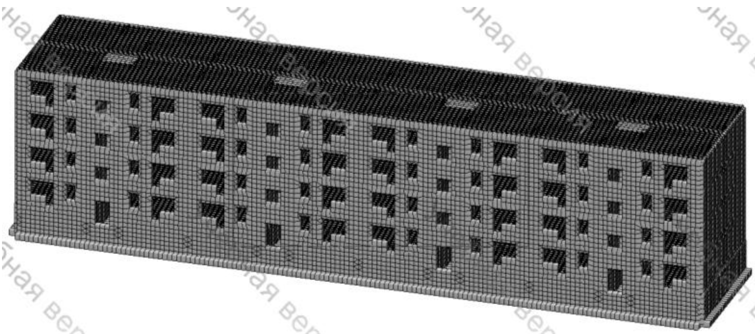

Fig. 1. General view of the series 310 building model in the form of 3D-visualization taking into account the element section dimensions

For the city of Samarkand seismic loads were assumed to be [2]: 8 points (periodicity of 500 years) and 7 points (periodicity of 150 years). Given that

seismicity calculations will involve the checking of sufficiency of load-bearing capacity of main walls as well as cast-in-situ reinforced concrete belts (binding) as their elements (separately, μ for these belts are not specified), the limit inelastic deformation was accepted to be $\mu = 2$ (t. 2.11 [2]). First 200 modes of building vibration were got for modal analysis. Vertical component of the seismic loads was allowed in accordance with the standards of KMK [2].

In accordance with KMK [2] with plane overall dimensions above 30 m it is necessary to take into account horizontal ground torsional vibrations relative to vertical axes. Given that the said building is generally symmetrical and its center of rigidity approximately coincides with the center of mass, the effect of horizontal ground torsional vibrations is calculated by multiplication of seismic load horizontal components by factor ξ, which is determined by the formula [2] (that was included in the calculation):

$$\xi = 1 + 0{,}4X/B = 1 + 0{,}4 \cdot 0{,}5B/B = 1{,}2, \qquad (1)$$

where X – the distance from the center of symmetry to an element in a direction perpendicular to the action of seismic loads; B - the size of a building in the same direction. The maximum of coefficient ξ is estimated in the formula (1) (when $X = 0.5B$).

For the initial estimation of brick wall stress state and choosing the most dangerous areas to check in detail there were selected indicative stresses related with appropriate brick wall checks in terms of seismic loads. As is generally known, such checks include calculations of off-centre compression, main tensile stresses and shear forces. Analysis of the application of indicative masonry stresses obtained in flat finite elements was well covered in the papers [3] and [4]. Walls load-bearing capacity was initially estimated at 7 points of seismic load intensity. Than the total force in the general section was measured in the most tense areas of the walls (N, M, Q) and masonry section load-bearing capacity was analyzed in accordance with the standards [5]. It was, as a result, found out that the most overloaded sections of the building walls at 7 points seismic load (with rare exceptions) had insufficient strength regarding the main tensile stresses both for the mortar grade M25 and that of M50 that will result later on in inclined cracks. Shear force strength (horizontal cracks) is not sufficient on the top floor due to small compressive stresses when the mortar grade M25 is used and is on the verge of exhaustion for grade M50. In case of the off-centre compression load-bearing capacity is always provided.

The results obtained correlate well with the data of full-scale inspections of residential 310 series buildings after the Tashkent earthquake [6] showing a significant (greater than 1 mm) crack openings at 7 points of seismic load intensity in the areas of main walls weakened by the openings. It should be noted that the failure to provide the main tensile stresses strength and shear forces strength does not mean the collapse of the structure (which did not happen to 310 series buildings even at 8 points of seismic load intensity and III

category of masonry seismic resistance). With this failure in strength regarding the main tensile stresses and shear forces there, as a rule, occurs a significant local damage of the particular separation wall or part of the wall involving the formation of inclined or horizontal cracks and subsequent inability to resist significant horizontal seismic forces (that are usually distributed between other less loaded structures, if available, that prevents the building final collapse.

The data obtained enable to conclude about an insufficient load-bearing capacity of the main masonry walls regarding the basic tensile stress and shear force at 8 points of seismic load intensity as well. However, the problem of ensuring the resistance to off-centre compression is yet to be cleared up. For this purpose, models with 8 points of seismic load intensity were used to estimate the basic most intensive compressive stresses, which amounted to N_3 = -1751 kN/m^2 for longitudinal walls and -1478 kN/m^2 for cross ones. The inspection of these wall parts based on the KMK formula involved the calculation of the exhaustion factor for the load-bearing capacity regarding the off-centre compression that amounted to 0.991 and 0.941, respectively [5]. A low reserve of resistance to the off-centre compression is the most dangerous, because exhaustion of compressive strength may result in a complete collapse of a wall overloaded part.

PC "LIRA-SAPR" was also used to estimate the sufficiency of monolithic ferroconcrete wall bands (binding). Automated selection of reinforcement revealed that a required diameter of longitudinal rods A-I or A400 (based on 4 reinforcement rods per section) with 7 points of seismic load intensity is 12 mm, in some areas – 14 mm. With 8 points – 14 mm, in some areas - up to 16 mm. Thus the existing band reinforcement of 4 rods A-I Ø10 mm is surely not sufficient.

An overall analysis of the stress state of masonry walls and there load-bearing capacity revealed many similarities with the results of inspections of 310 series buildings after the Tashkent earthquake [6 et al.].

To calculate and subsequently construct the reinforcement of the building under consideration, the seismic load intensity is assumed to be 8 points. In general, the basic options for the building earthquake resistance improving are as follows: direct strengthening of internal and external areas of the building walls with reinforced concrete "shirts" without cutting the building into sections and with a temporary resettlement of residents (option 1); direct strengthening of internal and external areas of the building walls with steel rolled elements without cutting the building into sections and with a temporary resettlement of residents (option 2); direct strengthening of internal and external areas of the building walls with Sika composite tapes without cutting the building into sections and with temporary resettlement of residents (option 3); direct strengthening of internal and external areas of the building walls with reinforced concrete "shirts" with cutting the building into sections an temporary resettlement of residents

(option 4); construction of reinforced concrete and steel elements for strengthening of accessible internal and external areas of the building walls without a complete evacuation of residents of the building (option 5). Limit inelastic deformation of reinforced walls is assumed to be μ = 4 (t. 2.11 [2]) as for the main complex building structures, except for the last option 5 (for unreinforced wall areas for which the value of μ remains to be 2).

Given that not in all the cases of the linear calculation the reinforcement elements provided unloading of the main structures down to the limiting point level there was carried out an iterative calculation involving some reduction in rigidity of overloaded parts. According to KMK [5] and earlier studies aimed at finding the total rigidity and deformation modulus of masonry structures in the limit state there were applied coefficients values 0.50…0.65 for the initial flexibility modulus E_0 that was used to take into account the decrease in rigidity of overloaded wall structures – individual groups of wall finite elements or damaged wall as a whole in reinforced building models.This approach is similar to the method of secant in ATC-40 [7].

The overall results of model calculations of the building, taking into account the reinforcement elements are presented in Table 1.

Table 1

The overall results of the stress state study of building models after reinforcement

Variant of strengthening	Degree of reduction of stress in the walls	Sufficiency of the reinforcement in the concrete of floor strapping	Presence of residual overloaded masonry elements
1	6,265	+	-
1	2,899	-*	+
3	2,089	-	+
4	5,153	+	-
5	4,833	+	+

* Note: Not critical in view of the presence of reinforcement steel rolled elements attached to binding.

Stress decrease in structures occurred both because of the transition of the load to the reinforcements, and because of the increase in the limit inelastic deformation of reinforced walls with μ = 2 to 4 (t. 2.11 [2]), and along with that (here and below) decrease in the reduction factor [2]:

$$r = 0{,}85\mu^{-0{,}67} = 0{,}854*2^{-0{,}67} = 0{,}537, \quad (2)$$

to the value r = $0{,}85\mu^{-0{,}67} = 0{,}854*4^{-0{,}67} = 0{,}337$, to 1,593 times.

For all the options there was assessed and confirmed the sufficiency of reinforcement strength.

It is worth noting that the occurrence of residual overloaded wall elements (see table 1) will not mean the collapse of building structures, because as the experience of earthquake up to 8 points shows the collapse of parts of Series 310 buildings does not occur, although the development of cracking and weakening is significant that drastically reduces the efficiency of their reconstruction. As noted above, the preventive reinforcement significantly reduces the stress level and the number of overloaded parts in the main walls. The preventive reinforcement is supposed to be more effective due to the following reasons: it prevents a significant damage and weakening of the main structures; substantially reduces the cost of the building rehabilitation after an earthquake and its aftershocks; greatly diminishes the risk of necessity to resettle residents after an earthquake.

In general, on the basis of the above we can draw the following conclusions. Multiple experimental studies of the stress state of the considered 310 series 4-storey residential building indicated that its structural system seismic resistance to an earthquake impact of 8 and 7 points is insufficient. The absence of complete destructions (even after an 8-magnitude earthquake in Tashkent) in building structures is explained by the redistribution of seismic effects to initially less strained partition walls and parts of the walls. It was found that in general, the construction of the concrete "shirts" in the main load-bearing structures (walls) is the most effective in terms of stress reduction (discharge) during seismic actions.

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