Строительные конструкции, здания и сооружения

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EXPERIMENTAL STUDIES OF LIGHTWEIGHT REINFORCED CONCRETE FRAME FOR SUSTAINABLE CONSTRUCTION

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Weight reductions, the use of natural materials, and ease of disposal are signs of environmental friendliness in building structures. This article studies designs with indicated features. It also provides the results of experimental studies of U-shaped reinforced concrete frames with pre-stressed reinforcement in posts and beam-column. The specific features of the frame construction are linear pre-stressed wire-rope reinforcement, connections of beam-column joint with a post, and connections with pre-cast reinforced concrete units bearing the moment of flection with foundation mat. The authors analyze the results of measuring the deformation capacity of such connections and their displacement due to vertical deal load.

Keywords: environmentally friendly building structures, pre-stressed reinforced concrete, testing of reinforced concrete frames, pre-fabricated reinforced concrete joints mobility, redistribution of forces.

Introduction

There exists a direction connected with improving framing and systems by means of increasing the degree of static indeterminacy and applying pre-stressed reinforcement [1–10]. One more direction for calculating the falling systems (limit equilibrium method) has been developed in theoretical studies as well as a direction where the determination of internal forces was based on the “bending moment – curvature” diagrams or the mutual angular deflection restricting the definite section of the rod system [2–4, 11].

There also exist experimental studies of U-shaped frames with non-tensioned reinforcement [12].

One study [13] presents experimental tests on continuous pre-stressed double-span beams. As a result of these tests the ratio of the span moment to the support moment (which according to elastic theory is equal to 0,61) changed to the experimental value of 0,75–1,28.

Some scientists [14, 15] on the base of theoretical and experimental studies make a conclusion about insignificant redistribution of internal forces in pre-stressed reinforced concrete constructions.

Other studies [10, 16–25] have reported about the influence of flexibility (deformability) of pre-cast rod structure joints. It has been found that flexibility of such joints in comparison to monolithic joints tends to increase [16, 17, 19, 22]. For instance, in these studies [17, 22] the angles of pre-cast joints were greater by 30–50 % and reached $12–44 \times 10^{-4}$, $70–90 \times 10^{-4}$ up to 12×10^{-3} rad.

The greatest challenge in improving constructions with pre-stressed reinforcement is the development of joints as far as it is necessary to combine the technology of reinforcement tensioning with the technology of bending moment transfer in the joint. The task is complicated if a more effective wire-rope reinforcement is applied.

While designing such systems the awareness of joint flexibility and taking it into account in calculations can become a problem.

Some results have already been published in the article [26]. The present article provides more extensive data on the base of additional experimental tests.

1. Experimental method (describing a pilot design)

External dimensions of a framed construction are the following (Fig. 1, 2): span length of 18 m, height of 9,35 m. The rod cross-section in the form of an I-beam: height of 1000 mm, the upper flange is 300×100 mm, the lower flange is 160×130 mm. The cross-section is variable: near the beam-column joint the cross-sectional height is 1000 mm, near the foundation – 600 mm. The flange had the same size – 300×100 mm. The wall thickness of the beam-

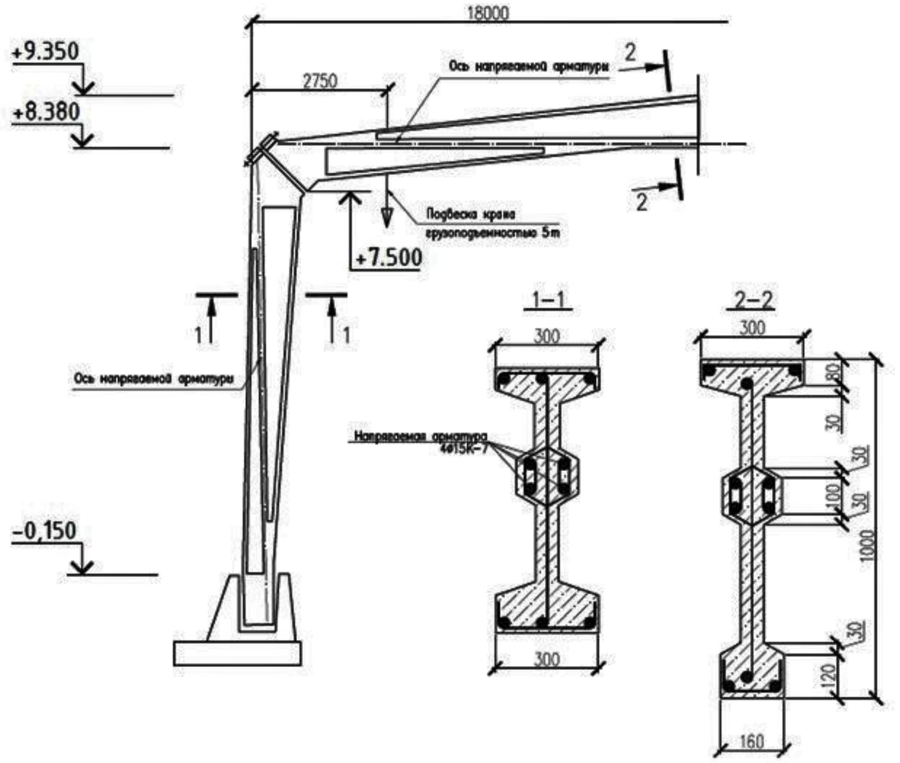

Fig. 1. Dimensions of the test structure

Fig. 2. Frame application

column joint and struts was 50 mm. The main reinforcement in the beam-column joint and struts was pre-stressed linear wire-rope reinforcement 4Ø15K7 according to GOST 13840, which was variable depending on the height of the section. In the beam-column joint this was done by its "fracture" with the formation of double-slope in the struts – by deflection from the flanges (Fig. 3).

The beam-column joint made the transmitting of the bending moment possible by means of the following construction peculiarities. Division into pre-cast elements (beam-column, struts) was performed by means of 45° oblique section (Fig. 3, 4).

The force in the compression area of this section was transmitted by welding of fixing metal parts (M2). The force in the tension area was transmitted by metal parts (M1), connected by 2Ø35 mm bolts. Parts M1 had anchors Ø25 of 500, 1000 and 1500 mm length, class S 400 (Table 1, junctions J1–J5), the pre-stressed reinforcement 4Ø15K7 had offset mechanical rope systems. The transmitting of tensile forces from pre-stressed reinforcement to Part M1 was performed

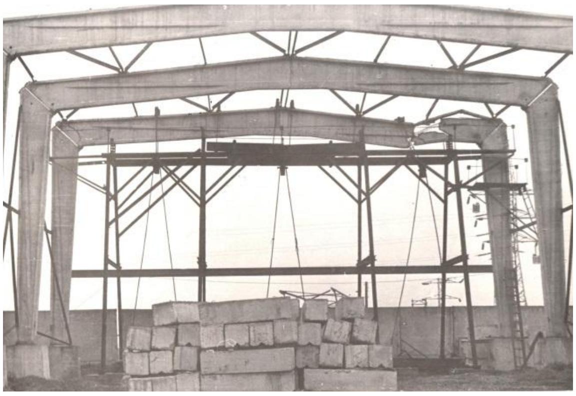

Fig. 3. Frame 1 after tests

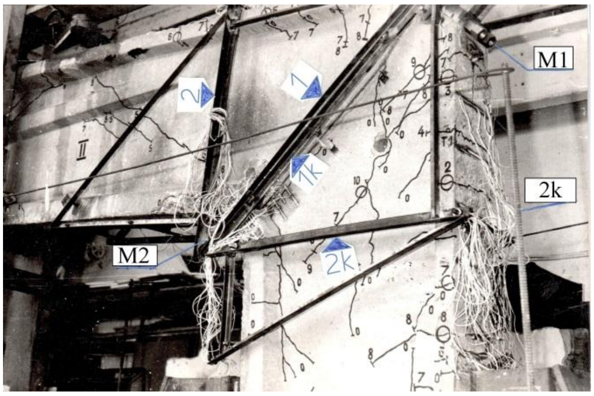

Fig. 4. Beam-column joint with struts. Flexibility of the joint is angular deflection 2 in relation to 2k

partially by these offset mechanical rope systems and by bond forces of concrete with anchoring rods and wire-rope reinforcement (lap splice). Junction J6 did not have offset mechanical rope systems on the ropes, anchoring rods $4\varnothing 25$, Class S 400, transmitted force to the rope reinforcement by lap splice.

In experimental frames (Fig. 3) we applied junctions J6. All the constructions were made of strength class C 25/30 concrete.

The joint of strut connection with ground base is designed as a combination of pre-cast column footing, monolithic grillage and four piles with the 400×400 mm cross-section. The column footing was connected to the grillage by means of welding fixing metal parts.

The above-mentioned constructions can be applied in industrial buildings of various purpose with under hung cranes, in public single-aisle and multi-span buildings with the use of medium columns of special design.

In comparison to beam-and-column constructions with beam-column joints hinged with struts this construction will make it possible to reduce material consumption: for a one-aisle frame – concrete by 20%, steel by 36%, for a double frame – concrete by 23%, steel by 33%. These constructions with some clarification in relation to movable load can be applied in bridge and trestle building (short span bridges with the carriageway on the level of stressed reinforcement).

The peculiarity of such constructions is a possibility to regulate the redistribution of internal forces artificially by means of creating displacements with the help of bolt tension in the beam-column joint with struts.

This type of regulating is performed during the construction phase and maintenance. In the latter case, it can be highly advisable, since in the course of time internal forces get redistributed. Changing the bolt tension, we can change the distribution of bending moments. This leads to the question connected with determining the optimal value of pre-stressed reinforcement [27] taking into account its changes in the internal forces redistribution process in use [13].

2. Methodology of experiments

We have tested two frames (Fig. 2) and six beam-column joints with a strut (Fig. 3), produced separately from the frame. In all the tests the constructions were loaded before destruction.

2.1. Testing two U-shaped frames

Loading of frames was performed by hanging weights (Fig. 3) with a gradual increase in their number (gradual static loading). Between the stages of loading the holding time was about 1 hour.

During load testing we measured the following:

– vertical displacements – bending flexures in seven points of the beam-column joint measured by bending meters with a scale interval of 0,01 mm (Fig. 5);

– horizontal displacements of Junction “A” measured by bending meters with a scale interval of 0,01 mm (Fig. 5a);

– elongation of bolts in the beam-column joint with the strut using a strain gauge pre-calibrated in the laboratory conditions. It allowed us to define the force in bolts, calculate the bending moment in the joint and define the corresponding distribution of moments at each stage of loading;

– angular deflection in a beam-column joint with a strut: angular deflection 1 and 2 in relation to 1k and 2k (Fig. 3) using special metal frames and bending meters with an accuracy of 0,01 mm;

– various angular deflections in Junction “B” (Fig. 5a) between the strut, column footing, grillage and grounding base using special metal frames with mechanical displacement measuring instruments, such as indicating gages and bending meters with a scale interval of 0,01 mm (Fig. 6);

– elongation and compression of metal parts connecting precast reinforced concrete column footings with monolithic grillages. Measuring was performed using mechanical strain gauges with an accuracy of 0,01 mm, which helped to calculate the forces and bending moments in these joints of the frames;

– edge deformations by means of indicating gages with an accuracy of 0,01 mm based on

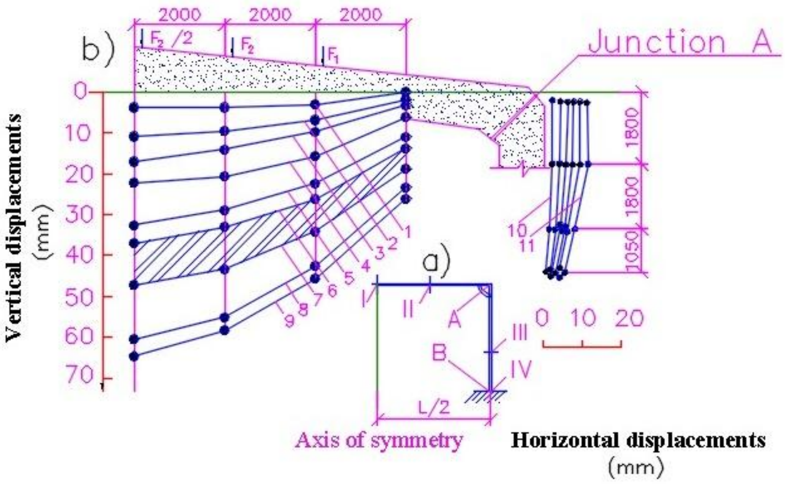

Fig. 5. Vertical and horizontal displacements of a beam-column and a strut:
a – scheme of a frame with the designation of sections and joints;
b – values of displacements in the process of increasing loading (when $\sum F_i$ correspondingly equals 1 – 35 kN; 2 – 160 kN; 3 – 216 kN; 4 – 272 kN; 5 – 328 kN; 6 – 346 kN; 7 – holding time during 10 hours; 8 – 404 kN; 9 – 458 kN; 10 – 160 kN; 11 – 544 kN)

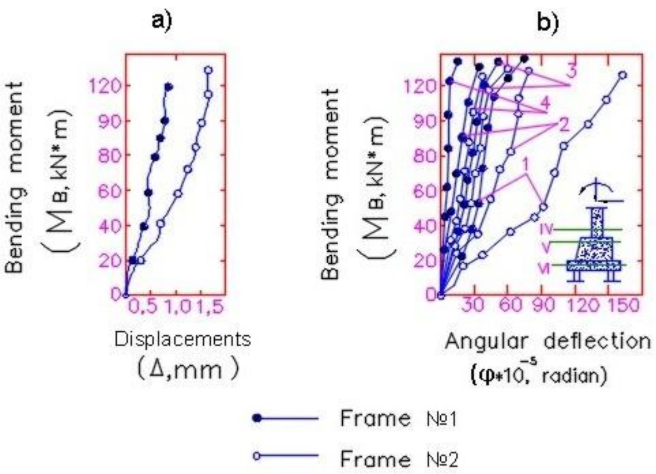

Fig. 6. Lateral (a) and bending (b) flexibility of Junction "B" in frames, where IV, V, VI– correspondingly to the cross-section of a strut, column footing, grillage: 1 – correspondingly to the ground level; 2 – IV in relation to V; 3 – VI in relation to the ground level; 4 – V in relation to VI

1000 mm in Frame 1 and 600 mm in Frame 2 in compressed and tensile areas in the following points: in the middle of the beam-column joint and at a distance of 4600–4700 mm from the middle, as well as in the strut at a distance of 3500 mm from the foundation edge.

These measurements allowed us to calculate angular deflections I, II, III (Fig. 5) and corresponding dependencies $M_i \sim \varphi_i$ (Fig. 7).

2.2. Testing of separate beam-column joints with a strut

Separately from the frames we tested beam-column joints with a strut in laboratory conditions using a special stand (Fig. 4). Loading was performed using a hydraulic jack as an application of concentrated force at the end of the beam-column section. The applied force place was defined from the condition of observing the proportion between the bending moment and lateral force corresponding to the elastic state. Loading was gradual with an interval of 45 min. On Fig. 8 shows the layout of strain gauges.

During loading of joints we performed the following measurements similarly to the joints in a frame (Fig. 4):

– deformations in pre-stressed wire-rope reinforcement and anchoring rods in the interaction zone (lap splice);

– distribution of deformations in the compressed zone of the joint;

– displacement value of pre-stressed wire-rope reinforcement in relation to part M1;

– lateral deformations in the interaction zone (lap splice);

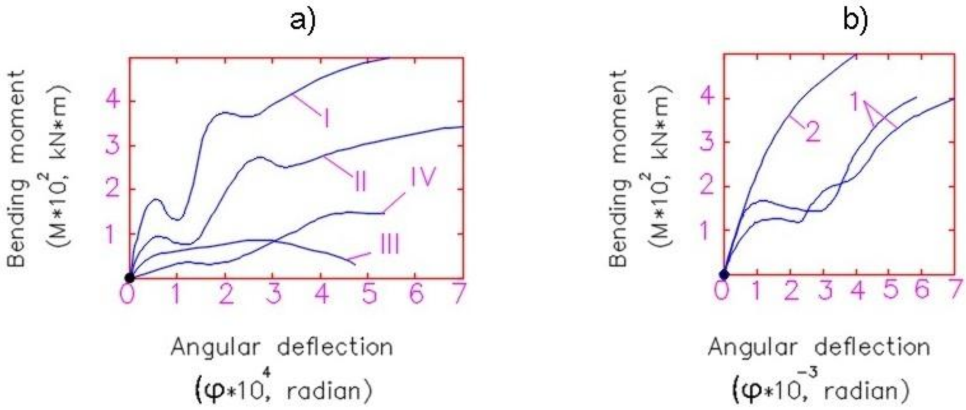

Fig. 7. Bending moment–angular deflection dependencies: a –parts of a beam-column and a strut in the cross-section zone in correspondence with Fig. 3a; b – flexibility of Junction "A" in correspondence with Fig. 3a (1 – in frames 1 and 2; 2 – samples of joints separate from the frame during laboratory testing in correspondence with Fig. 2)

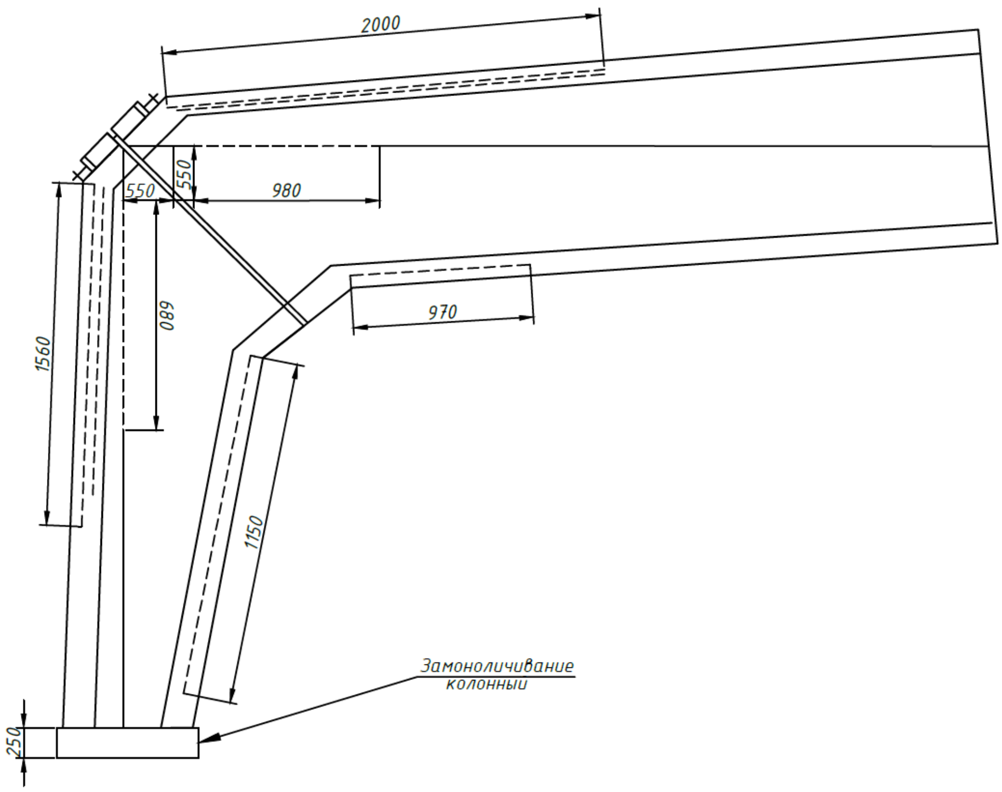

Fig. 8. Load cell arrangement diagram

– flexibility of cross-sections 1-1k and 2-2k as angular deflections;

– bolt deformations in the tensile area of the angles.

3. Experimental results

3.1. Results of joints testing

3.1.1. Mechanism of joints destruction

In the joints without offset mechanical rope systems the first fractures appeared in the lapping zone of reinforcement. With anchoring rods of 500 and 1000 mm length we could observe shifts of pre-stressed wire-rope reinforcement in relation to parts M1 and intensive fracture opening around edges of anchoring rods. With anchoring rods of 1500 mm length we did not observe such fractures, and the maximum loading was reached when anchoring rods demonstrated fluidity near Part 1, which was fixed by strain gauges as a sharp increase in deformations. Offset mechanical rope systems did not produce any effect on the destruction mechanism. One more reason for destruction was fluidity of anchoring rods near Part M1.

3.1.2. Interaction of wire-rope reinforcement with anchoring rods in the lapping zone

This interaction is peculiar due to the fact that reinforcements are anchored not in a monolithic concrete mass but in a concrete block, divided by fractures due to its stretching. This interaction was measured by means of axial deformations distribution graphs (Fig. 9).

Shear stress of cohesion along the reinforcement within each block was calculated according to the concrete-to-reinforcement bond formula:

$$\tau_{sh} = d_s \times E_s \times \Delta\varepsilon_s / 4 \times \Delta x, \qquad (1)$$

where d_s is reinforcement diameter, E_s – the elasticity modulus of steel, $\Delta\varepsilon_s$ – increment of relative reinforcement deformations on a section of Δx length.

For anchoring rods the calculated maximum stress of cohesion for tested samples of Junctions J3 and J4 was in the range of 1,4–1,56 MPa, while in the tested sample of Junction J6 it reached 1,82 MPa. The ultimate stress for Class C 25/30 concrete was 3,21 MPa and was calculated according to the following formula:

$$R_{sh} \cong 0{,}7\sqrt{R_b \times R_{bt}}, \qquad (2)$$

where R_{sh} is concrete strength when cutting or chipping, R_b – prism strength of concrete when compressing, R_{bt} – concrete strength when stretching.

The calculations proved that boundary strength of wire-rope and rod reinforcement is ensured by cohesion with concrete.

In this case, there were no longitudinal fractures in the lapping zone (according to the measurements of strain gauges fixed on the surface of concrete perpendicular to the direction of reinforcement).

On the basis of assessing the interaction of reinforcement in the lapping zone we can recommend a formula for defining the length of the lapping:

$$l_s = l_p + l_{an}, \qquad (3)$$

where l_p is the length of the area of transmitting preliminary tension, l_{an} – length of the area of rod reinforcement bolting.

Offset mechanical rope systems were installed on wire-rope reinforcement in order to determine their effect on the interaction in the lapping area and the destruction mechanism.

The destruction mechanism remained unchanged.

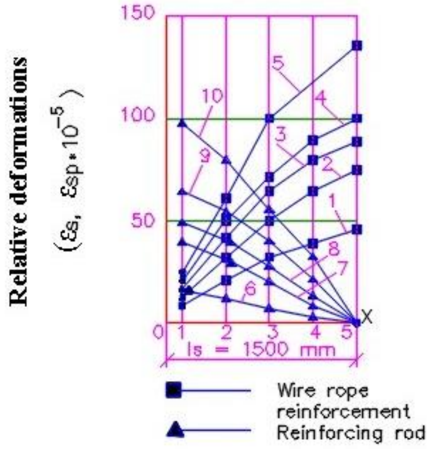

Fig. 9. Distribution of relative deformations according to the length of wire-rope (ε_{sp}) and rod (ε_s) reinforcements in Junction "A" under loading: 1 and 6 – 104 kN; 2 and 7 – 216 kN; 3 and 8 – 272 kN; 4 and 9 – 346 kN; 5 and 10 – 458 kN

The only difference was connected with the length $l_p \cong 0$. This created an equitable concrete prestress and led to the increase in fracture resistance by 1,75 times (in relation to the bending moment of fracture initiation) and by 1,4–2,1 times in relation to the width of fracture opening in the tensile area of the joint. The conditions of reinforcement interaction in the lapping area demonstrated that tension distribution in a wire-rope and rod reinforcement has become identical: the maximum value was observed around Part M1. Wire-rope reinforcement shifting resulted in the appearance of a fracture around Part M1 and fluidity of anchoring rods.

3.1.3. Fracture width analysis

Fracture width analysis is given for the joints with 1500 mm length of anchoring rods without the offset mechanical rope systems.

When the fracture width was 0,4 mm the share of loading in relation to the weight limit was 0,45–0,64. The data are presented in Table 1.

3.1.4. Flexibility of the joints

Flexibility was defined as angular deflections 1 and 2 in relation to 1k and 2k (Fig. 4).

Table 1

Main indicators of strength and fracture resistance of bolted joint specimens in the beam-column joint with a strut

Designation of junction specimens	Length of anchoring rods l_s, mm	Ultimate load F_{ult}, kN	Ultimate bending moment M_{ult}, kN× m	Fracture initiation F_{crc}/F_{ult}	Level F_i/F_{ult} at the fracture width a_{crc}		Level F_i/F_{ult} at the beginning of wire-rope shifting	Cause of destruction
					0,15 mm	0,40 mm		
J4	500	233	385	0,34	0,60	0,86	1,0	Shifting of wire-rope reinforcement
J3	1000	300	495	0,33	0,33	0,50	1,0	
J2	1500	560	902	0,27	0,36	0,64	0,5–0,6	Fluidity of anchoring rods
J5	1500	547	902	0,10	0,18	0,45	0,65–0,6	
J1	1500	550	–	0,20	0,27	0,54	–	
J6	1500	550	1100	0,45	0,63	0,90	0,82	

At the initial stage of loading the dependency between the bending moment and angular deflection was close to a linear one and was characterized as angular deflection 2 in relation to 2 k by values $8{,}3 \times 10^{-5}$ kNm/rad without offset mechanical rope systems and 16×10^{-5} kNm/rad with offset mechanical rope systems.

With the increasing bending moment, this dependency became nonlinear and was characterized as a ratio of nonlinear part of an angular deflection to the sum value of nonlinear and linear parts of the angular deflection $\xi \cong 0{,}48$ for all tested specimens of joints. At the same time during the holding time of constant loading during 25–30 min we could observe an increase in flexibility. The general flexibility can be presented by the following equation:

$$\varphi = \sum_{n=1}^{m} \varphi_n = \sum_{n=1}^{m} |\Delta_n| / h_0 \qquad (4)$$

where m is a number of factors influencing flexibility, Δ_n – edge deformations in a tensile and compressed areas between cross-sections 2 and 2k, which are formed due to bolt elongation, M1 part shifting, accumulation of deformations in the place of reinforcement interaction in the lapping area and crumpling of concrete under parts M2, h_0 – the distance between the measuring points Δ_n in a tensile and compressed areas (for Junction "A" $h_0 = 0{,}95\ m$).

3.2. Results of frame testing

In frame tests we applied Junctions "A" (Fig. 5) with anchoring rods $4\emptyset 25$ of class S 400 and 1500 mm length without offset mechanical rope systems.

Frame 1 got destroyed at stage 17 of loading at the total load of 54,4 tf (taking into account dead weight – 62,4 tf). Frame 2 got destroyed at stage 10 of loading at the total load of 45,81 tf (taking into account dead weight – 53,83 tf). The destruction took place at a distance of approximately 2 m from Junction "A" due to crushing of concrete in the horizontal part of the construction (Fig. 2).

Increasing displacements of beam-column joints with a strut points depending on the total load effect are shown in Fig. 5.

Thus we have found peculiarities of distribution and redistribution of bending moments along the beam-column and can demonstrate how fracture appearance and development influence them as well as flexibility of Junctions "A" and "B". For this purpose at each stage of loading we calculated moments at different cross-sections on the base of determining moments in Junctions "A" and "B" according to the indication of strain gauges; moment changes were compared to fracture formation and propagation.

The formation and propagation of fractures occurred according to the following scheme. Initially fractures appeared near section II. It can be explained by the fact that pre-stressed wire-rope reinforcement in this area is located near the gravity centre of the section. That is why the effect of concrete compression of the edging layers of this area got reduced whereas tensile stresses from external load increased. We have found this dependence with the help of the following analysis. In section II the ratio of bending moments $M_i/M_{ult} \cong 0{,}25$, and in sections I, III, IV 0,14; 0,06; 0,06 correspondingly and in Junction "A" it is 0,07, i.e. in section II the ratio of moments was the highest at the lowest degree of compression. The formation and propagation of fractures in section II led to the decrease of bending stiffness, accordingly to the decrease of bending moments in sections I, II and their increase in Junction "A".

In cases of load increase fracture formation began in Junction "A". After that, the increase in moments in Junction "A" stopped though moments in sections I, II increased. During later loading the redistribution of moments continued. This process is shown in Fig. 7, 10.

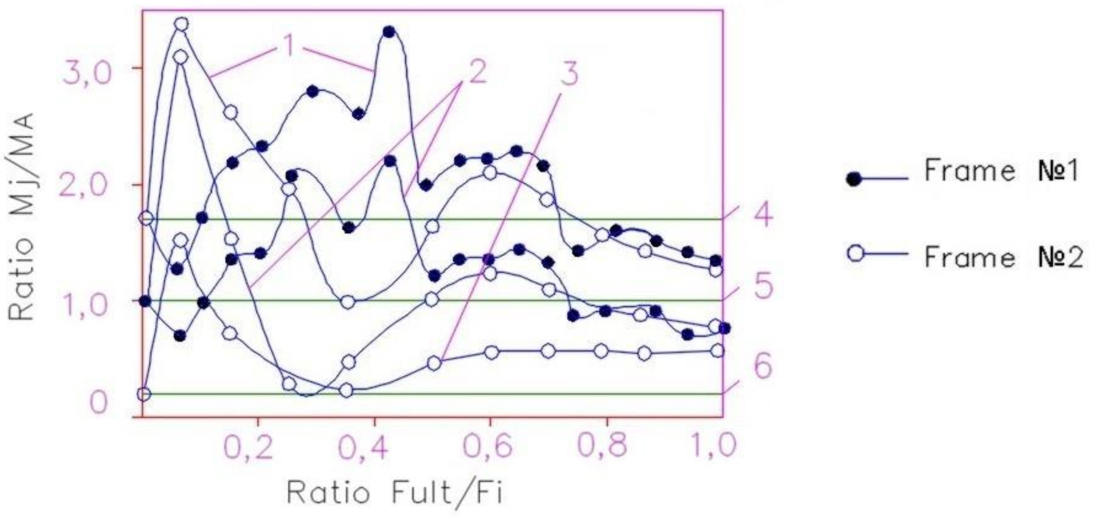

Fig. 10. Redistribution of bending moments in frames (designated according to Fig. 3a): 1 – ratio M_I/M_A; 2 – ratio M_{II}/M_A; 3 – ratio M_{IV}/M_A; 4, 5, 6 – from elastic analysis (at the initial linear flexibility from tests) correspondingly for sections I, II, IV

We can underline the following peculiarities:

– the greatest from the elastic deflection of bending moments distribution (taking into account initial flexibility of joints) happened at the beginning of loading and lasted as a variable process approximately till the operating level of loading around $0{,}6F_{ult}/F_i$;

– the deflection decreased above level $0{,}6F_{ult}/F_i$ and the process had a one-way character;

– the dependency "bending moment – angular deflection" in Junction "A" in frames (Fig. 7b) significantly differed from the similar dependency, obtained during testing of joints as separate elements (coincidence was observed only at the initial stage of loading).

The fracture analysis in frames has brought the following results. The maximum fracture opening by the moment of destruction was 0,05–0,70 mm in Junction "A"; 0,05–0,30 mm at section I; 0,10–0,40 mm at section II. When loading $F_i = (0{,}5 \ldots 0{,}6)\ F_{ult}$ fracture opening width was equal to 0,05–0,30 mm in Junction "A"; 0,05–0,10 mm at section I; 0,10–0,20 mm at section II.

During frame testing we determined the values of cross-sectional and bending flexibility of Junction "B" (Fig. 5a, b). Cross-sectional flexibility was understood as horizontal shifting under the effect of cross-sectional force of the whole foundation in relation to ground level. The reasons of bending flexibility were angular deflections, determined by deformability of embedding a strut in a column footing of the foundation (section IV in relation to V), deformations of a column footing and its fastening to the pile grillage (section V in relation to VI), pile grillage deflection in relation to ground level (section VI in relation to ground level). Bending flexibility of Junctions "A" and "B" are comparable here.

Conclusions

The construction of rod element joints creating static indeterminacy and forming lap splices of prestressed wire-rope and rod reinforcement can be called efficient according to the following:

– rod reinforcement reaches the yield point;

– fracture opening width at operating stages does not exceed 0,3 mm and can be regulated by the prestress value, the amount of rod reinforcement and installation of offset mechanical rope systems on wire-rope reinforcement.

We can observe alternate redistribution of bending moments determined by non-linear dependency "bending moment – angular deflection", which is formed during loading. In this regard accepting one way of calculating this dependency can be sharply questioned. That is why it is necessary to develop methodology taking into account the transformation of deformation dependencies while loading of statically indeterminate systems.

Since intensive alternate redistribution of internal forces occurs at an operational stage there arises one more question. Numerous systems in operation are subject to the effect of dynamic and pulsating load, thus alternate redistribution of internal forces can lead to the appearance of peak values in certain elements (sections) of the system and accumulation of damage reducing their lifespan. This issue needs further research in the future.

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ЭКСПЕРИМЕНТАЛЬНЫЕ ИССЛЕДОВАНИЯ ОБЛЕГЧЕННЫХ ЖЕЛЕЗОБЕТОННЫХ РАМ ДЛЯ ЭКОЛОГИЧНОГО СТРОИТЕЛЬСТВА

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Признаками экологичности строительных конструкций являются снижение их собственного веса, использование натуральных материалов, простота утилизации. Исследуются конструкции, обладающие указанными признаками. В статье приведены результаты экспериментальных исследований П-образных железобетонных рам с предварительно напряженной арматурой в стойках и балках. Особенности конструкции рамы: линейная предварительно напряженная канатная арматура, соединения узлов ригелей с колоннами, соединения сборных железобетонных элементов, воспринимающих изгибающий момент, с ростверком фундамента. Авторы анализируют результаты измерения податливости таких соединений и их смещения при вертикальной статической нагрузке.

Ключевые слова: экологичные строительные конструкции, предварительно напряженный железобетон, испытания железобетонных рам, податливость узлов, перераспределение усилий.

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