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EARTHQUAKE-RESISTANT REINFORCED CONCRETE FRAME OF SINGLE-STOREY BUILDINGS

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A B S T R A C T

The article presents the results of complex experimental studies of seismic resistance of one-storey reinforced concrete frames with developed hinge nodes of connection of cover structures with columns, as well as individual hinge nodes, under seismic-type loads.

In contrast to typical nodes, the use of hinge nodes in the design will ensure compliance of the design scheme with the real structural scheme, increase the dissipative properties of the frame during vibrations, due to energy dissipation in the hinge joints and save steel by eliminating the device of metal links in the cover structures, necessary for typical nodes. There is a copyright certificate for the invention (A.S.№1256222). There have been developed recommendations on calculation and design with an example of calculation.

1. DESTRUCTION NATURE OF ONE-STOREY FRAMES DURING THE SPITAK EARTHQUAKE

The consequences of strong earthquakes show that in one-storey reinforced concrete frame buildings the most vulnerable are the nodes of connection between columns and roof trusses. The nodes fail under strong seismic effects, leading to the collapse of buildings. Figure 1 shows some photos of the author based on the materials of operational survey of one-storey frame buildings in Leninakan (now Gyumri) during the 'Spitak' earthquake in Armenia, which occurred 07.12.1988) (Abakanov, 2009, 2012a).

The analysis of the causes of collapse of single-storey frame buildings showed that the applied typical nodes are not hinged, as it is traditionally accepted in the design calculations, and create a certain pinching of columns in the roof truss structures (Figure 1).

Figure 1. Types of destroyed one-storey frame buildings due to the destruction of the nodes of connection of rafters with columns in Leninakan city (now - Gyumri city)

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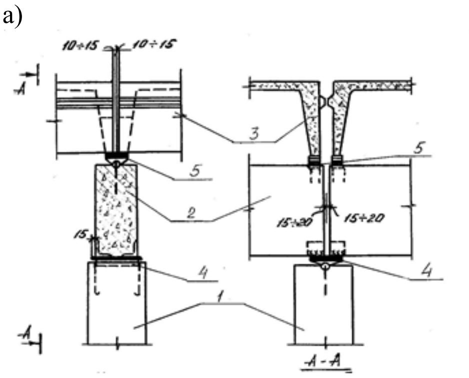
However, these factors have not been and are not taken into account in design practice. In addition, the use of typical nodes of connection of rafter structures with columns in buildings with design seismicity of 9 points, and in some cases with design seismicity of 8 and 7 points, require the mandatory device in the cover between the rafters of metal-consuming vertical links with struts.

2. HINGE ASSEMBLY

Before the Spitak earthquake in Armenia, the author had developed a new hinged joint for connecting columns with pavement structures. The Author's Certificate of Authorship (A.S. No. 1256222) was obtained (Abakanov, 1986). The design solution is based on the conditions of ensuring a clean hinge connection between the columns and the pavement structures, corresponding to the design scheme adopted in the design, Figure 2a.

In addition, this unit allows to increase the dissipative characteristics of the frame at vibrations due to energy dissipation in the hinge joints and to exclude the device of metal-intensive links in the coating and thus to obtain a significant saving of steel in the range of 3-5 kg. per 1 square metre of coating, as well as to reduce labour costs.

a)



b)

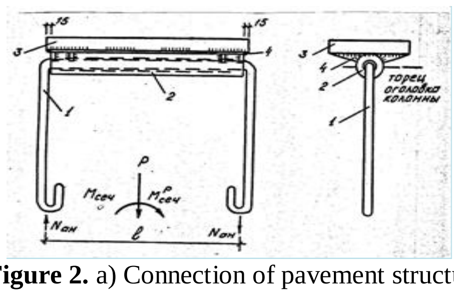


Figure 2. a) Connection of pavement structures with columns by roller joints: columns-1, rafters-2, pavement slabs-3, roller joints-4,5; b) Construction of metal roller joint assembly: arker core-1, sleeve-2, plate-3, stiffening ribs-4

The special feature of the hinge unit is that it provides a pure hinge connection of the pavement structures in one direction and a rigid connection in the perpendicular direction.

To obtain a hinge connection in two mutually perpendicular directions, the nodes in the pavement structures are installed at two levels - at the level of connection of beams with columns, with the hinge side along the axis of beams in the transverse direction of the frame, and at the level of connection of pavement slabs with beams, with the hinge side perpendicular to the axis of beams in the longitudinal direction of the frame (Abakanov, 2011).

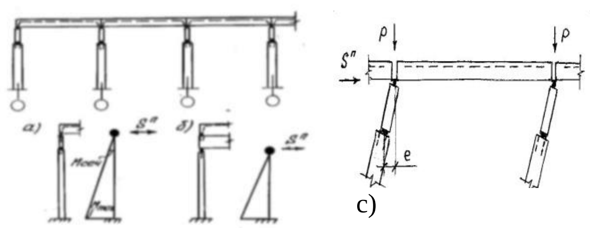
By design, the metal assembly is a roller joint consisting of a sleeve and a round anchor rod, which is also the core of the sleeve, as well as a metal support plate welded to the sleeve, Figure 2b.

The connection of the cover structures with each other is carried out in the following way. In the node (Figure 2a) of beam-column connection, the anchor part of the roller joint core (Figure 2c) is concreted in the column head, and the beam is supported on the metal plate, the embedded part of which is welded to the node plate.

At the joints between the cover plates and the beams, the roller joints are installed according to the same scheme, but with the hinge side facing away from the plane of the beams. In this way, a rigid connection is created at the joints of beams with columns in the direction from the plane of the beams, thus eliminating the arrangement of vertical metal links between the beams in the longitudinal direction of single-storey frame buildings.

3. CALCULATION DIAGRAM OF THE FRAME

The calculation diagram of a one-storey frame building with the use of a hinge assembly is assumed to be a single-mass cantilever system in the form of columns pinned in the foundations, Figure 3.



c)

Figure 3. Calculation diagram of the frame: a) in the longitudinal direction of the frame; b) in the transverse direction of the frame; *Sn* - seismic force; *Msec* - moment at the level of the hinge node in the direction of the rigid joint, longitudinal direction; c) deformed scheme of the frame in the longitudinal direction.

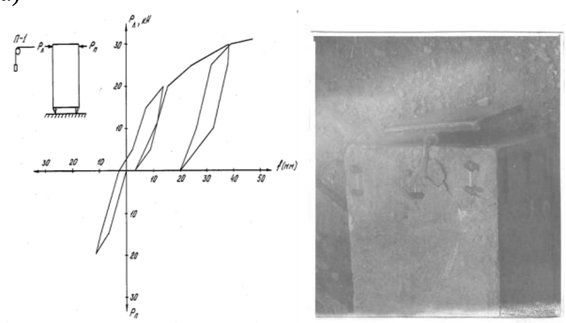
According to the design diagram, the height of the columns in the transverse direction of the frame is assumed to be equal to their actual height, because in the direction of the plane of the beams the connection between the columns and the beams is hinged, and from the plane of the beams in the longitudinal direction of the building, the design height of the columns is assumed to be greater by the height of the beam support. This is due to the fact that their connection is rigid and therefore the girder, as it were, is a continuation of the column in height up to the level of connection of the girder with the floor slabs.

4. STATIC LOAD TESTS ON COLUMN HEAD SECTIONS WITH PINNED CONNECTIONS AND METAL ROLLER HINGES

At the first stage, the column heads with steel hinged beam-to-column joint in full size (Figure 4a) were tested under seismic-type loads. The test results showed that the area of elastic operation of the nodes was 1.36 times higher than the design one, and the bearing capacity with regard to nonlinear operation was 2.73 times higher than the design one. At the same time, the shear stiffness of the node was high enough. In addition, the node has sufficient ductility, the ductility coefficient was 5.57, indicating that brittle failure of the node is excluded.

Table 1 shows the values of displacements of the top of the column heads (Figure 4a) and the angle of rotation at the level of the node, taking into account the effect of the longitudinal compressive load from the weight of the pavement on the operation of the node of connection of the rafter structure with the column.

a)



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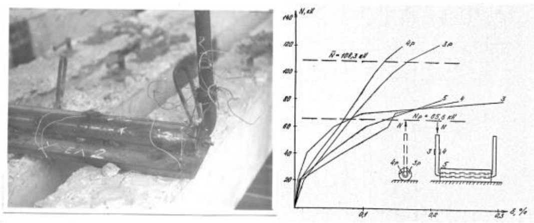

Figure 4. a) Testing of column caps with roller hinge assembly; b) Testing of the roller joint assembly

Table 1. Values of displacements and the angle of rotation

№ Of samples	N, kN	Displacement f, mm, rotation angle φ', minutes	Horizontal load P, in kN					
			10	15	20	30	35	45
№3	0	f	4,5	7,0	14,0	37,0	-	-
		φ'	20'	31'	1°1'	2°42'	-	-
№5	150	f	1,5	2,5	4,0	8,7	13,0	-
		φ'	7'	11'	18'	38'	1°57'	-
№6	150	f	1,7	2,5	3,8	6,9	8,9	13,1
		φ'	7,5'	11'	17'	30'	39'	1°58'

5. ROLLER JOINTS

The experimental ultimate load of the roller joints (Figure 4c) in the direction of rigid connection of beams with columns exceeded the design value on average by 1.6 times.

Thus, at the values of the design load determined by the normative resistances of the applied steel, the roller joints worked in the elastic stage of deformation, practically reaching the level of stresses in the sleeve and anchor of 80-90% of the yield strength, which can be seen from the deformation diagrams, Figure 4c. At the same time, the experimental destructive load exceeded the design load determined by the normative steel resistance by 1.8-2 times, and the design limit load determined by the steel time resistance exceeded by 1.1-1.2 times. The bearing capacity of the nodes with sufficient safety margin is reliably estimated by the developed calculation methodology.

6. DYNAMIC TESTING OF FRAME FRAGMENTS

In order to evaluate the performance of the assembly in real conditions, a large-scale model of the frame (Figure 5) and a life-size fragment of a single-storey frame building (Figure 6) were constructed and tested for the action of horizontal dynamic loads such as seismic loads using a powerful vibration machine B-3 (Abakanov, 2012b).

The results of dynamic tests of the fragment model showed that up to the load level of 20kN, the column deformation diagrams under static and dynamic loading are well matched, which can be seen from Figure 5c. The initial free vibration period of the fragment was T=0.20 sec and at the end 0.42. The logarithmic decrement of damping by the end of the tests increased by 2.5 times and was δ=0 .18.

a) b) c)

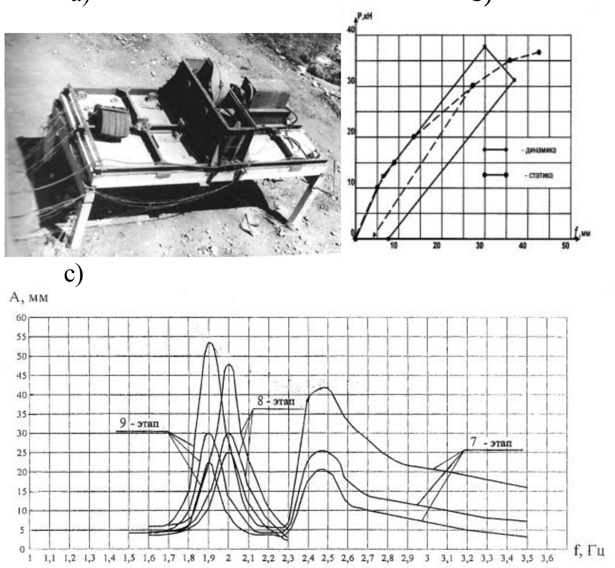

Figure 5. (a) General view of a fragment of the large-scale frame model; b) diagrams of the frame deformation: dynamic and static; (c) amplitude-frequency parameters from measured three points on the fragment cover, in the middle and on the edges of the cover

From the resonance curves (Figure 5c) it can be seen that with increasing load and number of oscillations the frequency decreased, torsional oscillation of the fragment in plan view occurred.

Table 2 shows the main experimental parameters of the fragment by stages of dynamic loading, according to which the restoring force (structure reaction) was determined using the differential equation of forced oscillations of the fragment under the design diagram of single-mass cantilever structure. Thus, the differential equation has the following form:

$$m\bullet\ddot{y} + r\bullet\acute{y} + R_y = Pt \qquad (1)$$

where Ry - fragment reaction, m-ÿ - inertial load determined from experimental data; Pt - disturbing force; r-ý= 2ε- m-ÿ - vibration resistance force; ε= δ/T - damping coefficient.

Table 2. Main experimental parameters of the fragment by stages of dynamic loading

№ of stages	ÿ, m/sec^2	Pt, kN	r•ý, kN	m• ÿ, kN	R_y, kN	δ	ε= δ/T
1	0.028	7,0	0,2	0,5	7,3	0,07	0,18
2	0.3	16,0	0,2	5,4	21,2	0,09	0,21
3	-	-	-	-	-	-	
4	0.67	17,0	0,3	12,0	-	-	
5	1.5	22,0	1,3	27,0	47,7	0,13	0,33
6	5.4	21,0	-	97,0	-	-	
7	7.28	22,0	3,7	131,0	149,3	0,09	0,23
8	4.74	26,0	-	84,6	-	-	
9	5.8	26,0	6,4	104,4	124,0	0,18	0,35

The calculated seismic load on the fragment at 9 point seismicity was 50kN. Starting from stage 6, the value of the experimental inertial load exceeded the design seismic load on the fragment by a factor of 1.9, at stage 7 - by a factor of 2.62 and at the last stage 9 - by a factor of 2.1.

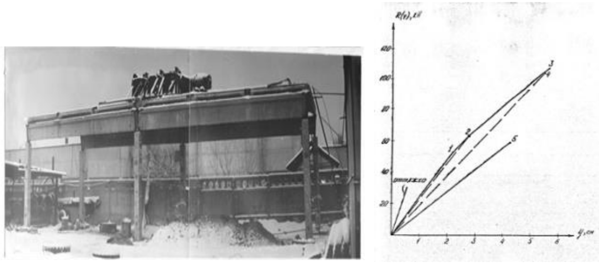

Figure 6. Life-size experimental fragment, deformation diagrams by stages of dynamic loading

Table 3 shows the main experimental parameters of the full-scale fragment (Figure 6) by stages of dynamic loading.

Table 3. Main experimental parameters of the full-scale fragment (Figure 6) by stages of dynamic loading

№ of stages	ÿ, m/sec^2	r•ý, kN	m• ÿ, kN	Pt, kN	R_y, kN	δ	ε= δ/T
Fragment retraction (6AI) - period Ts=0.74						0.086	0.116
I	1,07	2.75	50,22	6,0	53,47	0,131	0.170
	1,23	2.91	57,73	5,5	60,32	0,180	0.174
II	2,26	6.78	106,08	8,4	107,70	0,257	0.233
	2,21	7.11	103,73	8,2	104,82	0,257	0.233
III	1,20	5.14	56,33	7,5	58,69	0,257	0.233

The deformation diagram shows that the initial stiffness of the frame with increasing magnitude of the excited load gradually decreased with the formation and development of cracks in the columns up to 2.2 times at the last fifth stage of the dynamic tests. The logarithmic decrement of vibrations increased by 3 times, the frequency of resonant vibrations decreased by 1.4 times, and the period of natural vibrations increased by 1.5 times.

During dynamic testing of the fragment, a frame reaction of up to 107.7 kN was achieved.

The calculated seismic horizontal load on the fragment at an intensity of 9 points, determined according to the SNiP method was equal to 80.04kN. Thus, in the conducted experiments the value of dynamic load exceeded the design load at 9 point intensity by 1.5 times, and at 8 point intensity by 2.7 times.

7. RECOMMENDATIONS

The ‘Recommendations on calculation and design of earthquake-resistant single-storey reinforced concrete frame buildings with hinged joints between columns and roof structures, with an example of calculation’ were developed based on the results of complex experimental studies.

The main provisions of the calculation methodology are summarised as follows:

1. First, the seismic load Sn at the level of the floor slabs is determined according to the current norms, then the bending moment at the level of the connection of the column with the beam Mssection from the action of Sn, Figure 3.

$$M^{s}_{sec} = S^{n} \bullet h/n \,, \tag{2}$$

where M^{s}_{sec} – moment from the action of horizontal seismic load on the section at the level of beam-column connection node;
S^{n} - estimated seismic load acting at the level of the cover slabs resting on the beams;
h – height of the beam;
n – number of columns.

2. The bending moment in the section at the node from the weight of the vertical load, at the resulting eccentricity as a result of bending of the column is found by the following formula

$$M^{p}_{sec} = P \bullet e \,, \tag{3}$$

where M^{p}_{sec} – bending moment in the section at the node from the vertical load;
P – vertical load from the pavement weight taking into account seismic load per column;
e – eccentricity of the P application with respect to the original centreline of the column in the section at the node.

3. The eccentricity of the load P relative to the original centreline of the deformed column in the section at the node is determined by the following formula

$$e = e_{o} + e_{п.ф.} + e_{сл.}, \tag{4}$$

where E is the total eccentricity from the application of load P;
e_{o} – eccentricity resulting from column bending;
$e_{p.f.}$ – eccentricity arising from foundation rotation during building vibrations;
$e_{l.r.}$ - Eccentricity resulting from the roller hinge clearance under building seismic activity;
$e_{sl.}$ – random eccentricity, its value is taken not less than 0.03m. At the same time, the eccentricity arising from the column bending is determined by the formula

$$e_{o} = f_{1} – (H_{к} \bullet f_{1}/H_{o}) \,, \tag{5}$$

where f_{1} – displacement of the top of the column taking into account its increased height (H_{o} = $H_{к}$ + h); f_{1} is determined from the expression

$$f_{1} = S^{n} / C \,, \tag{6}$$

where C – frame column rigidity.
Eccentricity due to rotation of foundationsв

$$E_{p.f.} = O_{k.f.} \bullet H_{к} / 0{,}5L_{f} \,, \tag{7}$$

where $O_{k.f.}$ – settling (vertical displacement) of the edge of the foundation footing in the direction of the column bending during building vibrations; L_{f} – length of the foundation footing in the direction of column bending.

4. The force in the anchors of the roller joint as well as in the end sections of the bushing is determined based on the values of the total effect of bending moments M^{s}_{sec} и M^{p}_{sec}, arising in the cross-section at the node during building vibrations from forces S^{n} and P according to the diagram (Figure 3c) and the formula

$$N_{an.}=(M^{s}_{sec}/0{,}9\bullet L_{vt}+[(M^{p}_{sec}0{,}9\bullet L_{vt})–(0{,}5\bullet/2)], \tag{8}$$

where N_{an} - force in the anchor and in the end sections of the sleeve. The cross-sectional area (diameter) of the core anchor is determined from the tensile strength condition.

$$F_{p} = N_{ан} / R_{s} \,, \tag{9}$$

where F_{p} – cross-sectional area of the core anchor rod;
R_{s} - estimated resistance of the anchor material, in kN/cm2.

7. CONCLUSION

The design of the node was developed on the basis of analysis of the consequences of strong earthquakes with identification of the shortcomings of the applied typical nodes and their behaviour during strong earthquakes.

The hinge unit in one-storey reinforced concrete frame buildings ensures compliance of the design diagram with the real structural one, increases the dissipative properties of the frame during vibrations with energy dissipation in the hinge joints, and also reduces steel consumption due to the possibility of excluding metal links in the covering.

The results of static tests of metal roller joints as connecting elements of columns with beams showed the excess of experimental load-bearing capacity at its exhaustion over the calculation in 1.6-1.8 times.

Dynamic tests of the fragments of single-storey frames showed reliable operation and reliable assessment by calculation of the load-bearing capacity of the nodes at the level of impacts equal to the design seismic loads on the buildings.

The developed methodology of the nodes calculation allows to estimate and design the bearing capacity reliably and thus to provide reliable operation of the nodes under seismic loads of design intensity.

References

Abakanov, M. S. (2011). Single-storey frame buildings for seismic regions with articulated nodes of connection of cover structures with columns. Theses of reports. RNKS, IX Russian National Conference on earthquake-resistant construction and seismic zoning (with international participation), Sochi. The 6th-9th of September, -C.112-114. Retrieved from https://elibrary.ru/item.asp?id=19397305

Abakanov, M. S. (1986). Frame of one-storey earthquake-resistant building' A. S. 1256222. USSR. publ. 7.09.1986, Bul. no. 33 -4s: il. patenton.ru›patent/SU1256222A1

Abakanov, M. S. (2012a). Behaviour of single-storey frame buildings at Spitak earthquake, which occurred in 1988 in Armenia. Collection of abstracts of V World Congress of Engineering and Technology - WCET 'Science and Technology: step into the future', National Engineering Academy of RK, Almaty, 1st-2nd of, S-75-76. Retrieved from https://spec-expert.ru/reestr/abakanov-mirken-sejtkasymovich/

Abakanov, M. S. (2012b). Results of dynamic tests of fragments of one-storey frame buildings. Collection of materials of IV International Scientific and Practical Conference 'Construction in seismic areas', Almaty, the 16th-19th of October, P- 15-24. doi: https://spec-expert.ru/reestr/abakanov-mirken-sejtkasymovich/

Abakanov, M. S. (2009). Lessons of the Spitak earthquake that occurred in Armenia. Scientific and Technical Journal-№1, -C.15-21: Problems of Architecture and Construction. Samarkand, Uzbekistan: SamGASI. Retrieved from http://seismo-safety.info/abakanov-merken.html

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