Probabilistic Estimation Seismic Resistance of Plain Steel Frame

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Abstract. The purpose of the study is to estimate the coefficient $K_1$, which considers the permissible damage to buildings and structures in a probabilistic formulation. For the study, a flat steel frame was designed in accordance with SP 14.13330.2014 “Construction in seismic regions”. The frame calculation was performed in the LS-DYNA software package in a nonlinear dynamic formulation for a set of synthesized accelerograms. The set of accelerograms represents 50 implementations of random earthquake action with a fixed mathematical expectation of the dominant frequency. By gradually increasing the intensity of the action, the frame was brought to fracture in a numerical experiment, the results of which determined the safety margin of load-bearing capacity at each implementation. Further, by considering the safety margin and coefficient $K_1$, their average values were determined. The obtained results showed that the actual coefficient $K_1$ for the structure under study is significantly lower than its standard value, and the steel structure under consideration has a significant margin of seismic resistance. Thus, the developed method enables to estimate the actual value of the coefficient $K_1$.

1. Introduction

When designing buildings and structures in seismic areas, the linear-spectral method (LSM) of calculation is used. Strong earthquakes are very intense actions. In most cases, they result in damage to buildings and structures of varying degrees, in the structures there appear significant plastic strains, cracks and other defects. Structure behavior changes from elastic to inelastic. LSM is linear and does not allow to consider nonlinear strain behavior of structures directly. According to norms of different countries, when calculating by the linear-spectral method, to consider nonlinearities, a coefficient is introduced that reduces the earthquake action. In the Russian regulations on seismic-resistant construction [1] this is the coefficient $K_1$ considering the permissible damage to buildings and structures.

The coefficient $K_1$ was introduced in the norms of the first generations. It remains in the last edition of these regulations. However, the theoretical justification of this coefficient is available only for a limited class of simple structures. It is used for all buildings and structures designed in seismic areas.

The consequences of real earthquakes, such as the 1988 Spitak earthquake, showed that some structural diagrams of buildings and structures designed according to the norms of that time lack load-bearing capacity. One of the possible reasons for this is the insufficient relevance of the coefficient $K_1$ [2–6].
There have been little studies in this area. Only in a few papers [3,5,6] this problem was seriously considered from the viewpoint of the computational justification. Studies have revealed a real deficit of load-bearing capacity for some types of structural diagrams. However, these studies have not been widely developed, thus the topic remains relevant and important.

2. Problem statement
The purpose of the work is the probabilistic assessment of the actual value of the coefficient $K_1$ for the steel frame. The design diagram of the frame is shown in figure 1. Columns were assumed to be made of steel grade C345, beams – of steel grade C255 [7]. The steel behavior diagram was taken to be elastic-plastic (figure 2). Sections of columns and beams were selected using the PC LIRA 10.8 design complex in accordance with [1] in the form of rolled I-beams. The calculation was made for the 9-point earthquake design spectrum. The coefficient $K_0$ was assumed to be 1, $K_\psi$ was assumed to be 1.3, $K_1$ was assumed to be 0.25.

![Figure 1. The calculation scheme](image1)

![Figure 2. General view of steel deformation diagram](image2)

The frequency of the first mode of natural oscillations of the frame is 0.814 Hz.
For the calculation in a nonlinear dynamic formulation, 50 implementations of accelerograms of a nonstationary random earthquake action process with an average intensity of 9 points with a mathematical expectation of the dominant frequency of 0.814 Hz were synthesized. The simulation technique was adopted in accordance with [8-9]. One of the implementations is shown in figure 3.
Probabilistic calculation was carried out by the method of statistical tests. At each test, a deterministic frame calculation was performed for each implementation in the LS-DYNA software complex. Physical and geometric nonlinearities were considered in the calculation. The coefficient $K_1$ was assumed to be 1, $K_0$ was assumed to be 1. The damping parameter $\xi$ is 3% of the critical value.

If the frame did not collapse, an additional coefficient $K_c$ was applied to the accelerogram acceleration values. And the calculation was repeated. Thus, the coefficient increased in increments of 0.1 until there was a partial or complete collapse of the frame. The maximum value of the coefficient $K_c$ is the desired safety margin of the load-bearing capacity (seismic resistance).

![Figure 3. One of the accelerogram implementations: a - accelerogram; b - spectral composition](image)

3. Results and discussions

As a result of the calculation, the load-carrying capacity safety margins $K_c$ were obtained for all implementations. Tables 1 and 2 show the coefficients obtained for partial and total collapse, respectively.

Table 1. The load-carrying capacity safety margins $K_c$ for partial collapse

| № of implementation | 1   | 2   | 3   | 4   | 5   | 6   | 7   | 8   | 9   | 10  |
|----------------------|-----|-----|-----|-----|-----|-----|-----|-----|-----|-----|
| Safety margin        | 3.4 | 3.6 | 3.4 | 3.2 | 3.3 | 3.0 | 3.6 | 3.4 | 2.8 | 3.2 |

| № of implementation | 11  | 12  | 13  | 14  | 15  | 16  | 17  | 18  | 19  | 20  |
|----------------------|-----|-----|-----|-----|-----|-----|-----|-----|-----|-----|
| Safety margin        | 3.9 | 2.5 | 3.2 | 3.0 | 3.0 | 3.0 | 5.0 | 2.9 | 2.9 | 3.1 |

| № of implementation | 21  | 22  | 23  | 24  | 25  | 26  | 27  | 28  | 29  | 30  |
|----------------------|-----|-----|-----|-----|-----|-----|-----|-----|-----|-----|
| Safety margin        | 3.0 | 2.6 | 2.7 | 3.2 | 3.5 | 4.9 | 4.7 | 5.2 | 3.1 | 3.0 |

| № of implementation | 31  | 32  | 33  | 34  | 35  | 36  | 37  | 38  | 39  | 40  |
|----------------------|-----|-----|-----|-----|-----|-----|-----|-----|-----|-----|
| Safety margin        | 3.0 | 3.8 | 1.9 | 3.7 | 4.2 | 2.3 | 3.7 | 2.8 | 3.5 | 3.7 |

| № of implementation | 41  | 42  | 43  | 44  | 45  | 46  | 47  | 48  | 49  | 50  |
|----------------------|-----|-----|-----|-----|-----|-----|-----|-----|-----|-----|
| Safety margin        | 3.0 | 5.8 | 3.4 | 3.9 | 3.3 | 3.9 | 3.9 | 3.5 | 3.2 | 2.9 |
Table 2. The load-carrying capacity safety margins $K_c$ for total collapse

| № of implementation | 1   | 2   | 3   | 4   | 5   | 6   | 7   | 8   | 9   | 10  |
|----------------------|-----|-----|-----|-----|-----|-----|-----|-----|-----|-----|
| Safety margin        | 3.4 | 3.6 | 3.6 | 3.2 | 3.5 | 4.1 | 3.6 | 3.4 | 2.9 | 3.2 |
| № of implementation  | 11  | 12  | 13  | 14  | 15  | 16  | 17  | 18  | 19  | 20  |
| Safety margin        | 3.9 | 2.5 | 3.2 | 3.9 | 3.0 | 3.0 | 6.6 | 2.9 | 3.1 | 3.1 |
| № of implementation  | 21  | 22  | 23  | 24  | 25  | 26  | 27  | 28  | 29  | 30  |
| Safety margin        | 3.0 | 2.6 | 2.7 | 3.2 | 3.5 | 5.2 | 5.5 | 5.6 | 3.1 | 3.0 |
| № of implementation  | 31  | 32  | 33  | 34  | 35  | 36  | 37  | 38  | 39  | 40  |
| Safety margin        | 3.0 | 4.1 | 1.9 | 4.1 | 4.2 | 2.3 | 4.2 | 2.8 | 3.7 | 4.5 |
| № of implementation  | 41  | 42  | 43  | 44  | 45  | 46  | 47  | 48  | 49  | 50  |
| Safety margin        | 3.1 | 6.4 | 3.4 | 4.1 | 3.3 | 3.9 | 3.9 | 3.6 | 3.2 | 3.2 |

Based on these results, histograms for the safety margin were constructed (figure 4, 5) and the average values of safety factors were determined. In the case when the partial collapse of the frame was taken as the fracture criterion, the average safety margin was found to be 3.42. In the complete collapse it was 3.6.

![Figure 4. Histogram for the safety margin for partial collapse](image1)

![Figure 5. Histogram for the safety margin for total collapse](image2)
Using the obtained safety margins, it is possible to determine the actual coefficient $K_1$, by dividing the design value by $K_c$. Thus, the actual values of the coefficients for the structure under consideration are 0.073 and 0.069, respectively.

In [6, 10, 11], the seismic reduction factor for steel structures was not estimated. In [3, 5], both steel and reinforced concrete buildings and structures were studied. For the considered steel structures, the actual coefficient $K_1$ also turned out to be less than the design one (0.2). However, this coefficient was obtained in the deterministic formulation and for the spatial design diagram.

In [12] the zipper-braced frames are explored, but the design scheme was different from that considered in this article. In [13] steel frames were considered, but the reduction coefficient was estimated during earthquake recurrence.

4. Conclusions
The study showed that a flat steel frame, designed according to standards [1], considering the coefficient $K_1=0.25$, has a significant safety margin of load-carrying capacity (seismic resistance). Depending on the implementation of the random accelerogram, the seismic resistance safety margin varies from 2.3 to 6.6. Analysis of the study results shows that the load-bearing capacity of structures depends not only on the design solutions and the calculated intensity of the construction site, but also directly on the accelerogram parameters.

Since the study was conducted in a probabilistic formulation, it enabled to estimate the average value of the seismic resistance safety margin, and thus the coefficient $K_1$. The results showed that the obtained value significantly differs from the standard. With the help of the developed methodology, it is possible to clarify $K_1$ for the main types of structural solutions for buildings and structures. This, in turn, enables to obtain a significant economic effect for some types of structures designed in seismic areas (for example, for buildings with a steel rigid frame) while ensuring the required level of reliability.

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References
[1] SP 14.13330.2014 Construction in seismic regions, The updated edition of SNiP II-7-81*, Moscow, 2014.
[2] O.V. Mkrtchyan and S.V. Bulushev, Actual problems of earthquake engineering, "Loleyt readings-150". Modern methods of calculation of reinforced concrete and stone structures by limit states, pp. 270-278, 2018.
[3] O.V. Mkrtchyan and G.A. Dzhinchvelashvili, Accounting Problems of Nonlinear Seismic Stability in the Theory (Hypothesis and Errors), Moscow: MGSU publ, 2012.
[4] O.V. Mkrtchyan and G.A. Dzhinchvelashvili, Fundamental principles of earthquake resistance calculation to be reflected in the next generation regulations, MATEC Web of Conferences, vol. 86, 01017, 2016.
[5] O.V. Mkrtchyan, G.A. Dzhinchvelashvili, and M.S. Busalova, Normative approaches to structural design calculations in a non-linear framework, MATEC Web of Conferences, vol. 86, 01018, 2016.
[6] A.V. Sosnin, About refinement of the seismic-force-reduction factor (K1) and its coherence with the concept of seismic response modification in formulation of the spectrum method (in order of discussion), Bulletin of Civil Engineers, 60(1), pp. 92–116, 2017.
[7] SP 16.13330.2011 Steel structures. The updated edition of SNiP II-23–81*, Moscow, 2011.
[8] O.V. Mkrtchyan and A.A. Reshetov, Method for determining baseline characteristics most unfavorable accelerograms for linear systems with finite number of degrees of freedom, Vestnik MGSU., № 8. pp. 80-91, 2015.
[9] O.V. Mkrtychev and A.A. Reshetov, A representative set of earthquake accelerograms for calculation of buildings and structures on seismic effects, Vestnik MGSU, 12, № 7 (106). pp. 754-760, 2017.

[10] G.M. Blanco, C.L. Buron and Z.F. Salas, Estimation of seismic force reduction factor in reinforced concrete frame buildings, Revista de Obras Publicas, vol. 165, issue 3603, pp. 36-41, 2018.

[11] E.V. Muho, G.A. Papagiannopoulos and D.E. Beskos, A seismic design method for reinforced concrete moment resisting frames using modal strength reduction factors, Bulletin of Earthquake Engineering, vol. 17, issue 1, pp. 337-390, 2019.

[12] A.J. Vaseghi, A.M. Esmaeilnia and B. Ganjavi, Ductility reduction factor for zipper-braced frames, European Journal of Environmental and Civil Engineering, vol. 22, issue 11, pp. 1341-1363, 2018.

[13] G. Abdollahzadeh and A. Sadeghi, Earthquake recurrence effect on the response reduction factor of steel moment frame, Asian Journal of Civil Engineering, vol. 19, issue 8, pp. 993-1008, 2018.