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Earthquake engineering optimization of structures by economic criterion

Оптимизация конструкций сейсмостойких сооружений по экономическому критерию

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Abstract. The article deals with an economic optimization problem of structures designed on areas exposed to seismic hazard. Profit (cost-effectiveness) from building usage due its design life-cycle is considered as objective function in this optimization task. Building damage state evaluation procedure and repair cost estimation method are proposed in this study. A criterion and a variable parameter of an optimization problem is suggested here as well. There is an algorithm, which combines seismic computation results with economic performance indicators of damage state. The example of practical use of the algorithm is shown with the help of numerical simulations and economic parameters analysis for industrial building frame designed using different seismic retrofit schemes. Financial costs for each seismic retrofit scheme of a building are determined based on cost estimates, which allows to obtain the near-real estimation of seismic retrofit cost and financial losses from repair works of injured structural elements after various earthquakes.

Аннотация. В данной статье на примере промышленного здания рассмотрена проблема оптимизации конструкций, проектируемых в сейсмически опасных районах. Решается задача оптимизации, за критерий оптимальности в которой принимается экономическая эффективность, при этом исследуется её изменение на протяжении всего жизненного цикла здания в зависимости от определённых варьируемых параметров, принимаемых на этапе проектирования. Полученный алгоритм оптимизации основывается на ранее предложенных методах оценки экономического эффекта сейсмостойкого строительства, доработанных и развитых, а также приведённых к структуре, позволяющей решать конкретные инженерно-экономические задачи сейсмостойкого проектирования. Практическое применение алгоритма продемонстрировано на примере расчёта экономической эффективности антисейсмических мероприятий при выборе того или иного варианта несущего каркаса здания, которая оценивалась на разных временных этапах его срока

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службы. Рассматривается подход по определению суммарного конструктивного ущерба, который может быть нанесён зданиям различными землетрясениями за весь ожидаемый срок службы. Также приведены некоторые методы по определению подобного рода ущерба, рассматриваемого как результат нелинейной работы материала элементов здания. Финансовые затраты на возведение каждого варианта здания определены на основании сметных расчётов, что позволяет получить близкое к действительности представление о стоимости строительства и ремонтов после землетрясений.

Introduction

The article deals with the algorithm to optimize design projects of earthquake resistant buildings by an economic criterion, which is subject to research.

Optimization based on economic criterion is a widespread type of optimization. Such problems can be generalized as follows: choose an option at the lowest cost out of varied options for design structures to meet certain requirements. In most cases it is about how to minimize weight, dimensions and labor intensity. Most recently a notion of life-cycle, including service life of a building, has started to be commonly used [1, 2]. In case of earthquake-resistance structures optimized by cost-effective indicators it is necessary to consider changes of its state (failure, partial failure, repair) due to seismic impacts, i.e. consider life cycle of a building. The article is concerned with the method to choose an optimal project solution for an earthquake resistant building based on the analysis of cost to profit ratio during the whole life cycle of a building.

The problem under consideration has been intensively discussed in the different studies over the last 15 years. Optimization problem by an economic criterion in earthquake engineering has been defined in [2, 3]. There are suggestions how to perform economic analysis of prospective seismic damages in these research works as well. There is a number of approaches how to identify defects and damages in buildings, changes in service characteristics, idle period in the articles [4–10]. The works [1, 2] are also concerned on this problem. It is important to mention the research work [11], where the problem of economic damage forecast and seismic structural optimization were considered for the first time.

However, the algorithm which combines seismic estimation results with economic performance indicators of damage value has not been suggested. Optimization criteria have not been specified to full extent. The algorithm to vary structural characteristics suggested in these works is complex and time consuming. The task of the present article is to eliminate these drawbacks. And the authors objective was to develop such an optimization procedure (criteria, variables and algorithm) which could correspond to the design provisions and codes specified for earthquake engineering in Russian Federation.

Methods

Optimization problem. Objective function

Economic criterions suggested in [1] is considered as the most algorithmically solvable. These criterions are based on the ideas developed in [11]. The proposed in this study optimization criterion makes it possible to combine two different economic parameters developed in [1]. The criterion suggested below makes an optimization procedure more visual form a physical standpoint. When the construction site is located on areas exposed to seismic hazard, structural analysis must be provided with consideration of seismic load. Therefore, extra activities are stipulated to sustain seismic retrofit of the frame in accordance with the computation results. If there is an earthquake due the life-cycle of the building it will cause some damages, and the damage in the retrofitted frame after earthquake will be less than in the frame without seismic retrofit. That is to say, if to compare the frame with seismic retrofit and the one without it, a prevented damage will be found, but some damage will be caused anyway. Thus, when the number of anti-seismic measures are considered along with the one, which excludes these measures, the damage can be shown as a sum of two components:

$$D(I)=D_{pr}(I)+D_{rel}(I), \tag{1}$$

where D_{pr} – the prevented damage, D_{rel} – the real damage, D – the damage that could occur in the frame without retrofit, I – the intensity[1] of earthquake.

[1] Earthquake intensity is scaled by MSK-64 in terms of a rate.

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Damage estimations can be explicated in the following way. Let's assume that there are n types of structural elements (columns, trusses, beams and bracing elements) which may fail. If the frame is not retrofitted, all these structural elements fail (or partially fail). If the frame is retrofitted, then just a few structural elements fail (or partially fail). Consequently, the cost of structural members which have not failed is the prevented damage D_{pr}. The cost (repair costs) of structural elements which have failed (or partially failed) is the real damage D_{rel}.

Having modified the formula (5.10) proposed in [1] with consideration of Eq. (1) for calculation the cost effectiveness of the manufacturing building during the considering life-cycle equal to N years we obtain the economic effect

$$E_{eff}=\left[\sum_{n=1}^{N} P(n) - K_{build}\right] - K_{ant} - f(k,N)\sum_{I=I_{min}}^{I=I_{max}} L(I)\cdot\Big(D(I) - D_{pr}(I)\Big), \tag{2}$$

where $P(n)$ – profit (total by N year); K_{build} – investments into construction of the building (cost of the entire building including costs to equip manufacturing lines); K_{ant} – seismic retrofit cost (if there are no such costs the building will fail due to seismic impacts and there either will be no profit or minor profit as damages will limit production output); $f(k,N)$ – cost adjustment factor in accordance with recommendations given in [1] under the formula $f(k,N)=\left(\frac{1}{k}-1\right)[1-(1-k)^N]$. Here $k=\frac{d+d^*}{1+d}$, where d^* – depreciation rate (the parameter which determines reduction of the building value over the time inverse to its maintenance period) d – annual profitability of production; $L(I)$ – average number of rate I earthquakes on construction site; N – time after the maintenance start (years).

In the formula (1) $D\text{-}D_{pr}=D_{rel}$ the real damage as well as the prevented damage contain the following:

- repair and replacement cost of injured structural elements;
- losses of the equipment inside facility;
- losses in profit due to idle period when repairing.

Since the manufacture must be financially justified the value E_{eff} must be positive. The Eq. (2) is clear. If there are no anti-seismic measures taken then K_{ant} = 0 and D_{pr}=0. Thus financial losses due to earthquakes are determined by the damage $D(I)$ taken into account a number of earthquakes with intensity I and damage costs adjustment from year to year. If anti-seismic measures have been taken the real damage $D(I)_{rel}=D(I)-D(I)_{pr}$ is less than $D(I)$ but on the other hand financial losses increase owing to K_{ant}. Correlation of all these variables determines optimization E_{eff}.

If we keep in the Eq. (2) only those variables which depend on anti-seismic measures then we obtain

$$E=-K_{ant}+f(k,N)\sum_{I=I_{min}}^{I=I_{max}} L(I)\cdot D_{pr}, \tag{2}$$

which corresponds to the formula (5.9) in [1]. Thus introduction of the variable D made it possible to demonstrate that two different (as it is stated in [1]) approaches to compute cost-effectiveness of anti-seismic measures are identical as a matter of fact.

In economic optimization problem the variable E should be taken as an objective function. An optimization criterion in the formula (3) is a certain one and allows for seismic characteristics of a construction site. And we cannot say the same about the criteria suggested in the article [2].

Optimization problem. A variable. Constraints

Taking into account standards and regulations of the Russian Federation [12] and design practice based on the linear and spectral theory and the concept which considers plastic deformation of buildings using the reduction factor K_1 regarding seismic loads then while optimizing a project the factor K_1 should be taken as the variable. With such a variable there is no need to vary structural members (beam cross sections, columns, coefficient of reinforcement, etc.). While setting the value K_1 there is an opportunity to automatically select these variables using the design software (SCAD Office, Ing + MicroFe, SOFiSTiK and others). Requirements of standards are automatically met as well. In other words the value set for K_I determines an option of seismic retrofit. As to previous reasons and formulas it is apparent that it is not always the case when reduction of K_1 factor in total reduction costs (it is often assumed as a norm) since reduction of capital costs is correlated with increase in repair costs.

A constraint matrix in a variation problem is a constraint matrix for estimation results in accordance with standards and regulations.

It is assumed that the frame keeps its structural stability due the seismic ground shakings [13–17]. If the requirement is met it secures life safety and no serious injuries for people inside a building due the seismic event. That is why the criterion (3) does not include costs associated with rehabilitation of injured people and casualties.

Financial loss determination

Suggestions to determine financial losses corresponding to the earthquake with the intensity should be considered as the weakest point in the methods meant for optimization of the seismic building design suggested in [1, 2]. As a matter of fact this problem is avoided in these articles owing to a formal introduction of different definitions and notions. Naturally, the problem stated can be solved regarding only one certain structure, and not as a general case. However the algorithm which combines engineering computations and financial indicators can be suggested.

Nowadays the nonlinear static Pushover analysis is recommended to be used for nonlinear analysis in earthquake engineering by a major part of regulations. The method is based on the use of the curve describing load-bearing capacity of a structure [13, 15, 17]. This curve depicts dependence of the structure roof displacement and the base shear. It is suggested to use assumptions, software and methods that is considered by the structural engineer as adequate to obtain this curve. In the Pushover method this curve is transformed into the spectrum describing structural capacity which correlates roof displacements and seismic accelerations of the structure. If we determine the damage state as a function of the roof displacement then we will obtain an opportunity for analysis of successive destruction of structural elements with the increasing of seismic load. Said another way, we can obtain the building performance objective as a function of seismic shaking intensity. This performance objective is actively used in regulations and guidelines [13, 15–17] in foreign seismic structural design practice. It represents a certain damage state or performance level of a building after a certain earthquake, which is applied at the stage of a new building design or seismic rebuilding operations of the existing one. For instance, the chapter 3 of ATC-40 [17] (Applied Technology Council of California Seismic Safety Commission) is concerned to the problem of building performance objective selection based on standard performance levels. This approach is named “performance based earthquake engineering” (design of structures with specified seismic characteristics). It should be noted that damage state of load-carrying structures are classified in national guidelines of structural reliability assessment [4].

We can do a successive transformation of the capacity curve having plotted financial indicators of damage loss for different values of roof displacement instead of performance characteristics. We can name such a curve as financial curve**.** Having applied the nonlinear static analysis to the seismic structural behavior we can suggest the following algorithm to compute the cost-effectiveness of seismic retrofit schemes:

1. Set the intensity of seismic ground motion I.

2. By the value I determine maximum accelerations A.

3. With the help of pushover determine maximum roof displacement of a structure u.

4. By the value u, using financial curve**,** determine the damage D_{rel}.

5. Using I and D_{rel}, make computations under the formula (3).

Computation of displacement u can be provided in different ways, which a designer considers to be sufficient regarding accuracy of source data. For instance, with rough estimations it is possible to apply an energy method [6].

There is a wide range of tools to create the capacity curve:

- increment nonlinear analysis using FEM in ANSYS, SOFiSTiK etc.;

- extended finite element method (XFEM) for reinforced concrete structures [18];
- pseudo-stiffness method [19];

and others.

Calculation example. Numerical analysis

A. Load-bearing framing description

The building, considering as the example in this study, is a one-story rectangular shaped frame with plan dimensions $43.0 \, x \, 108.0$ m and the height equal to 10.5 m. A two-story outbuilding with office and utility rooms is arranged inside the building. Roof coverage consists of truss rafters spaced at 4 m and mounted on supporting trusses which are assembled on columns through pillar sections. Top and bottom truss chords are unbraced with horizontal longitudinal girders, diagonal girders are set between top chords. Trusses are designed as 24 m span with 2 sloping surfaces and 12 m span with one sloping surface and uniform triangular grid with lowering diagonals. Supporting truss are designed 24 m span and 12 m with horizontal parallel chords and uniform triangular grid with lowering diagonals. Top chords of supporting truss ST2 and ST6 as well as pillar sections are designed from steel C345, all the rest metal structures are made from steel C245. Framing columns are precast reinforced ones, column section К1 – $600 \, x \, 600$ mm, columns К2 – $400 \, x \, 400$ mm. Strength grade of concrete B25 and reinforcing bars A500 are taken for columns. The layout for load-bearing structures is shown in the Figure 1.

B. Project types and computations

Computation is executed for three options[2] of load-bearing framing: typical S_{tip}, partially-reinforced Frame S_{PS} and maximum reinforced Frame S_{MS}. A finite element model of the frame was developed using SCAD Office and a seismic load was set in accordance with requirements [12]. The first type Frame S_{tip} is a building designed for a basic combination of design loads, dead weight and anticpated live loads without any seismic considerations. Sections for steel elements and column reinforcement are set in accordance with the computation results.

A fragment of analytical model with the results of reinforcement computation for columns which are not vertically braced are shown in the Figure 2. The bending moment value at the foundation level of these columns achieves the peak, therefore, the area of longitudinal reinforcement required to be the maximum. Color scale depicts total area of reinforcement bars (cm^2) placed along longitudinal axis of a bar next to the edge of a member section under ultimate tension.

The second type S_{PS} is represented as the frame of the same configuration but designed for rate 8 earthquake with the coefficient $K_1=0.5$. The task of the coefficient K_1 correction in the linear spectral method, which is the basis for this computation, have been considered in the works [20–23]. Seismic retrofit is confined the strengthening of load-bearing structures by increasing the area of longitudinal reinforcement in reinforced columns and increasing sections of steel structures elements section in accordance with computations.

The fragment of the analytical model with the results of reinforcement computation for columns, which are not vertically unbraced, are shown in the Figure 3.

The third type S_{MS} is the maximum reinforced frame. The coefficient $K_1=1$ is taken numerical simulations. The design seismic shaking intensity corresponds the rate 9 earthquake in this case. Seismic retrofit is executed as it is done for the option S_{PS} by increasing the section areas of longitudinal reinforcement of the reinforced columns and increasing sections of steel structures elements to a greater extent. Reinforcement in sections of the reinforced columns is selected with no regard crack resistance.

[2] Option of load-bearing framing implies the seismic retrofit scheme.

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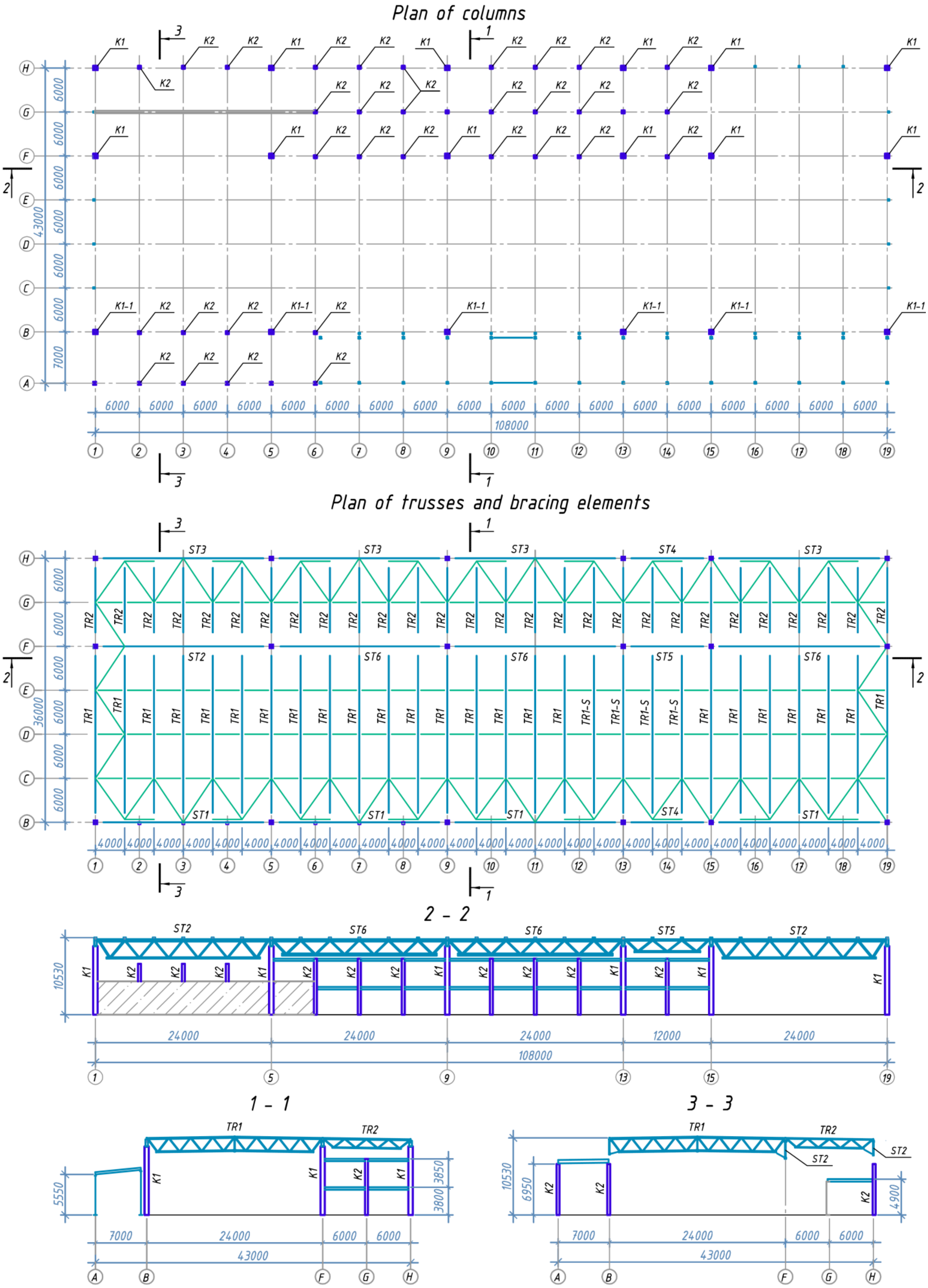

Figure 1. Layout for load-bearing structures

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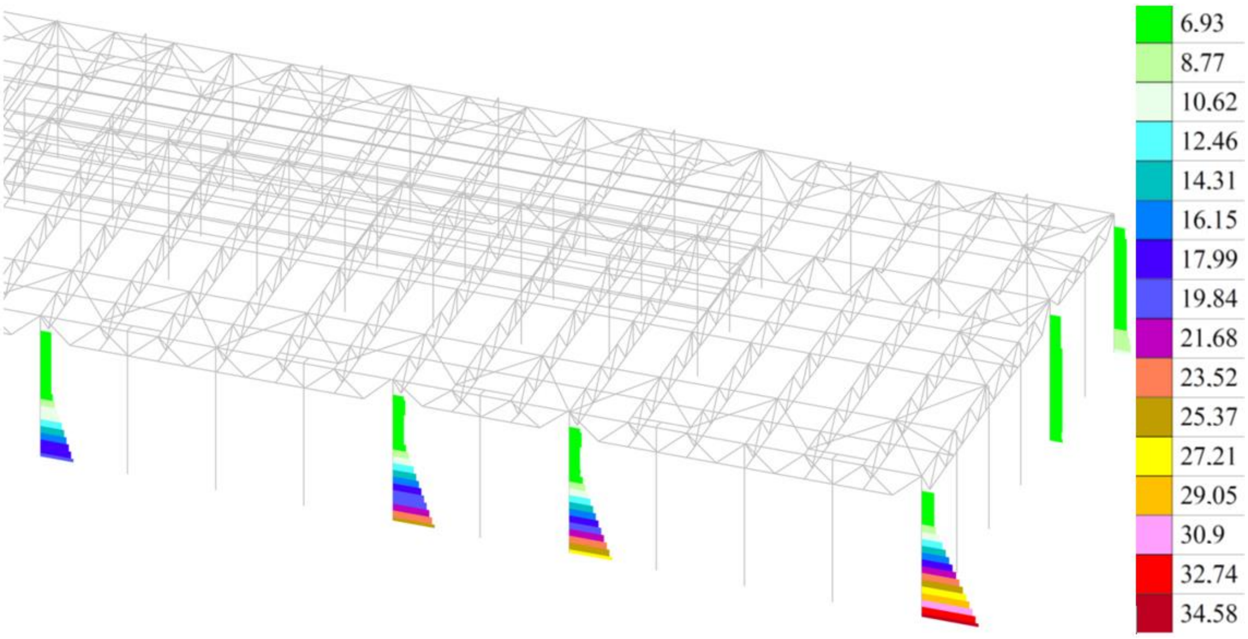

Figure 2. Reinforcement colored distribution diagram in sections of the reinforced columns members of Frame S_{tip}

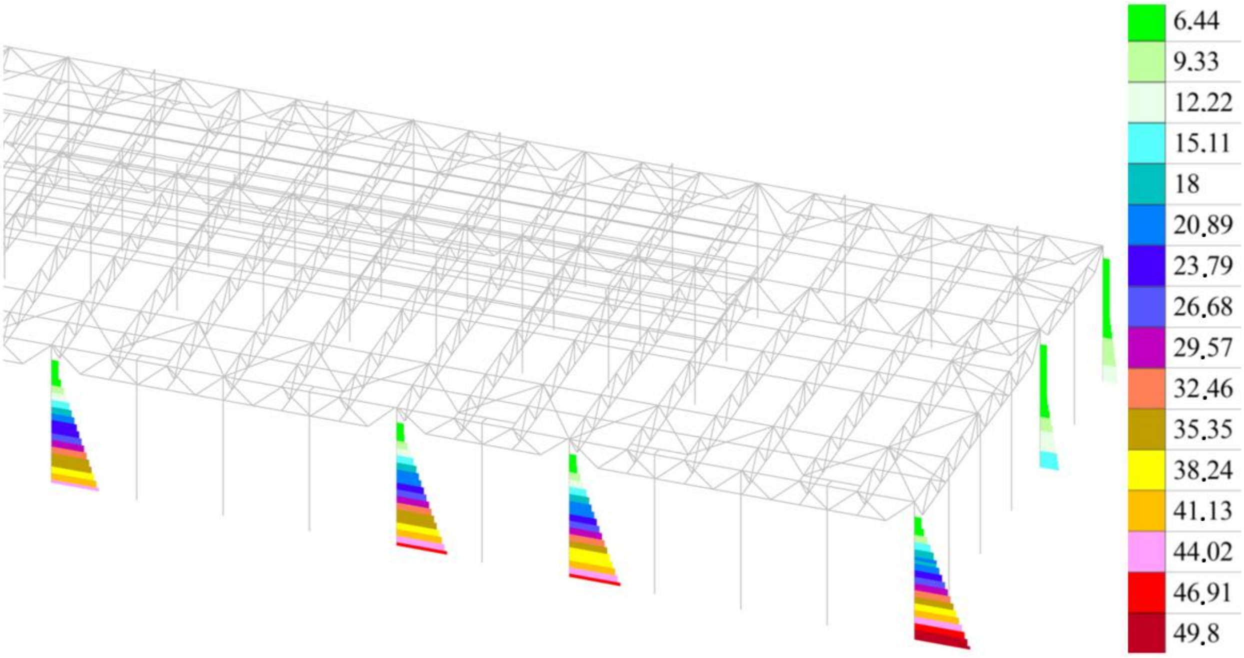

Figure 3. Reinforcement coloured distribution diagram in sections of the reinforced columns members of Frame S_{PS}

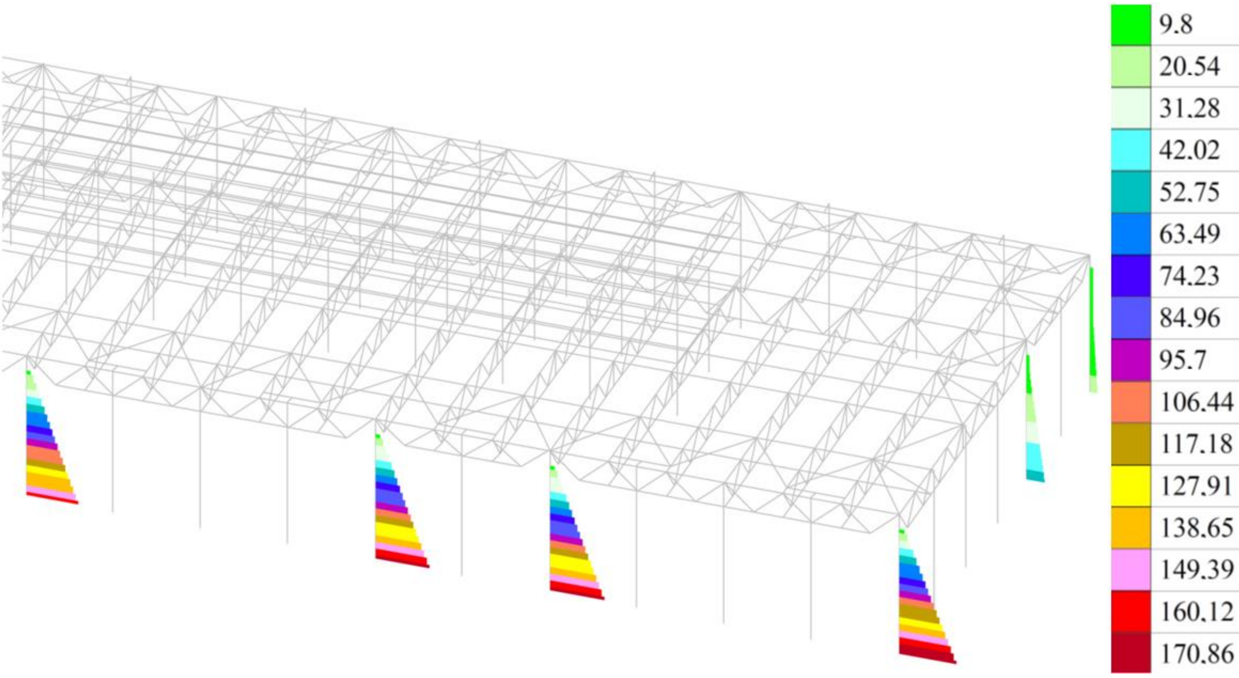

Figure 4. Reinforcement coloured distribution diagram in sections of the reinforced concrete columns members of the Frame S_{MS}

On the basis of these computational data three options of structural frames were designed for this object, technical specifications of steel were worked out, specification worksheets considering steel consumptions were elaborated. Data for reinforced concrete columns sections obtained for each option are shown in the Figure 5.

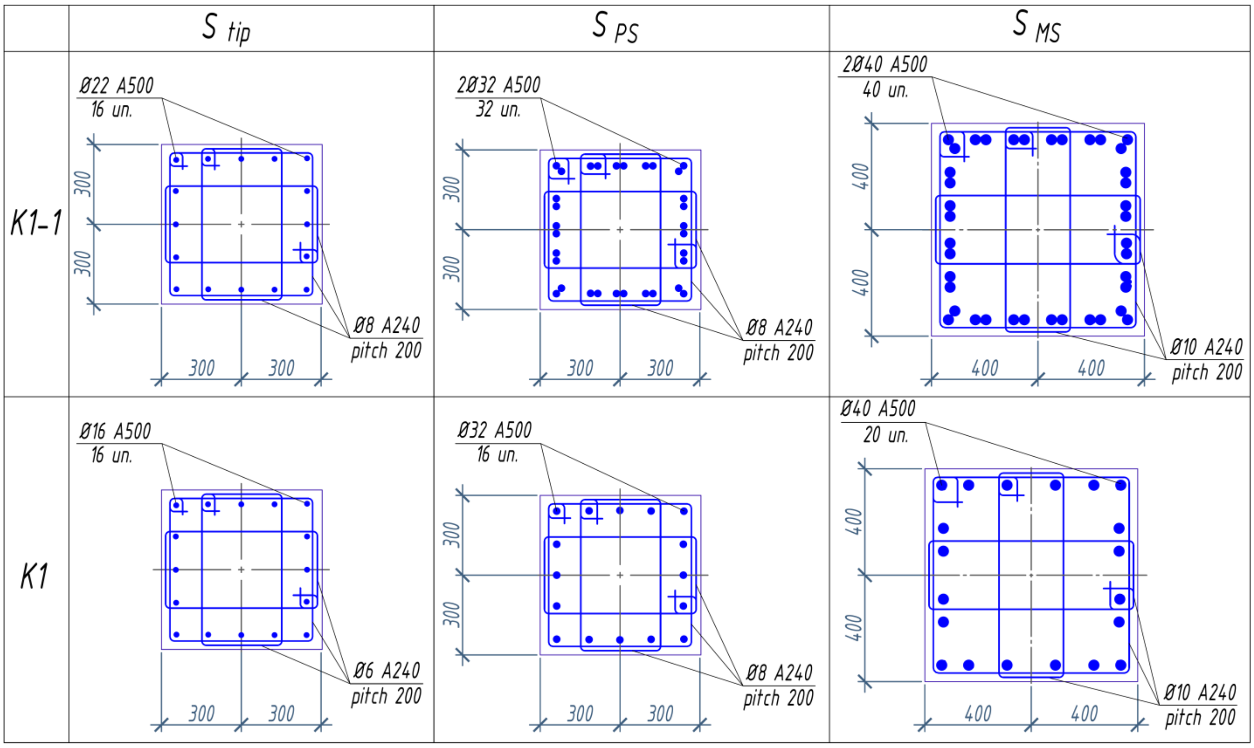

Figure 5. Data on reinforcement for sections of the reinforced concrete columns K1, K1-1

C. Costs of load-beraing framing works and seismic retrofit measures

Costs of structural framing works regarding a manufacturing building C based on technical specifications C were estimated for each of these three options. Costs associated with framing works for the option S_{tip} is equal to $C_{tip}=15.9$ mln rubles. Framing costs for the option S_{PS} are equal to $C_{PS}=20.19$ mln rubles, and framing costs for the option S_{MS} are equal to $C_{MS}=30.8$. A contractor allocates investments in the amount of $C_{tot}=200.0$ mln rubles to cover total construction costs. This value is taken as 1, and then we obtain

- Frame S_{tip} costs on the per-unit basis is equal to $C_{tip}=\frac{15.90}{200.0}=0.08$;
- partially reinforced Frame S_{PS} costs on the per-unit basis is equal to $C_{PS}=\frac{20.19}{200.0}=0.101$;
- maximum reinforced Frame S_{MS} costs on the per-unit basis is equal to $C_{MS}=\frac{30.80}{200.0}=0.154$;

Then costs associated with seismic retrofit for each of two options can be determined under the formulas:

- $K_{ant,\,PS}=C_{PS}-C_{tip}=0.101-0.08=0.021;$
- $K_{ant,\,MS}=C_{MS}-C_{tip}=0.154-0.08=0.074.$

D. Buildings capacity curves creation

As it was noticed before it is suggested to take assumptions, computation methods and software, which a designer considers to be appropriate, to get a capacity curve (which describes building capacity under load). Hereafter, a certain approach is suggested to create this curve based on the example of the building under consideration. The approach implies estimation of horizontal shear at the base (Base shear) and horizontal displacement of the building's top (roof displacement) and creation of the curve based on the data obtained.

A number of points to create the capacity curve according to the guidelines [17] should correspond to the number of performance levels set due the object design, which is relevant to the idea of performance based earthquake engineering, what is mentioned in [2, 5, 22]. Thus, the performance objective of this building is based on 2 requirements:

1) When the rate 6 and smaller earthquakes occur the frame should behave elastically, and its maintenance should not be terminated ("Operational" performance level in accordance with the guidelines [17]);

2) When the rate 9 earthquake occurs failures of load-bearing structures should not exceed ultimate ones, which may cause a collapse failure of the frame; the building should be suitable for repairs; life safety and no injures should be secured ("Life Safety" performance level in accordance with [17]).

Capacity curves should be developed for each frame. Firstly, computation for the option S_{MS} – the building with the maximum reinforced load-bearing framing – should be done. The value of horizontal base shear, which corresponds to the first point, is suggested to be determined by exerting lateral force on longitudinal load-bearing column – truss joints, which is not going to cause inelastic behavior of any structural members

$$V_1=\sum_{i=1}^{K} V_i,$$

where V_1 - horizontal base shear, which corresponds to the first point of capacity curve;

V_i – value of the lateral force exerted on the point of intersection between an *i* column and roof truss;

K – the number of main load-bearing columns supporting the roof structures (18 columns).

Lateral forces V_i are set in SCAD, then linear computation is executed and reinforcement for column sections is selected, which value after a number of iterations aimed at setting the value V_i should be close to the data in Figure 5 to the greatest extent. In accordance with the method indicated for the option S_{MS} the value of the lateral force V_1 = 5827 kN is obtained, under which action a bending moment at the base of columns is taken up by the reinforcement close to its elastic behavior. Displacement of the structure's top Δ_{roof1} corresponding to its base shear is equal to 0.094 m.

The value of the horizontal base shear which corresponds to the second point is determined on the following assumption: after a maximum bending moment in the column under maximum load is reached inelastic deformations will start to emerge and develop, and it will pull up all the rest columns due to force redistribution. It is necessary to sum up all the ultimate bending moments in the load-bearing columns and divide this sum by the column height taken in the Figure 1.

$$V_2 = \frac{\sum_{i=1}^{K} M_{pr,i}}{h}$$

where V_2 – horizontal base shear, which corresponds to the second point of the capacity curve;

$M_{pr,\,i}$ – the value of the failure bending moment at the base of an *i* column;

K – the number of basic load-bearing columns supporting the roof equal to 18 (for columns К and К-1 the values of the failure bending moment will be different);

h – column height.

According to computations we obtain $V_2 = 11085$ kN. The value Δ_{roof2} can be found multiplying by 2.2, then $\Delta_{roof2} = 0.207$ m. Based on these data we may create the capacity curve for the option S_{MS}. Points 1 and 2 for the option S_{PS} and S_{tip} can be found in a similar way, corresponding capacity curves are created on the basis of these points shown in Figure 6.

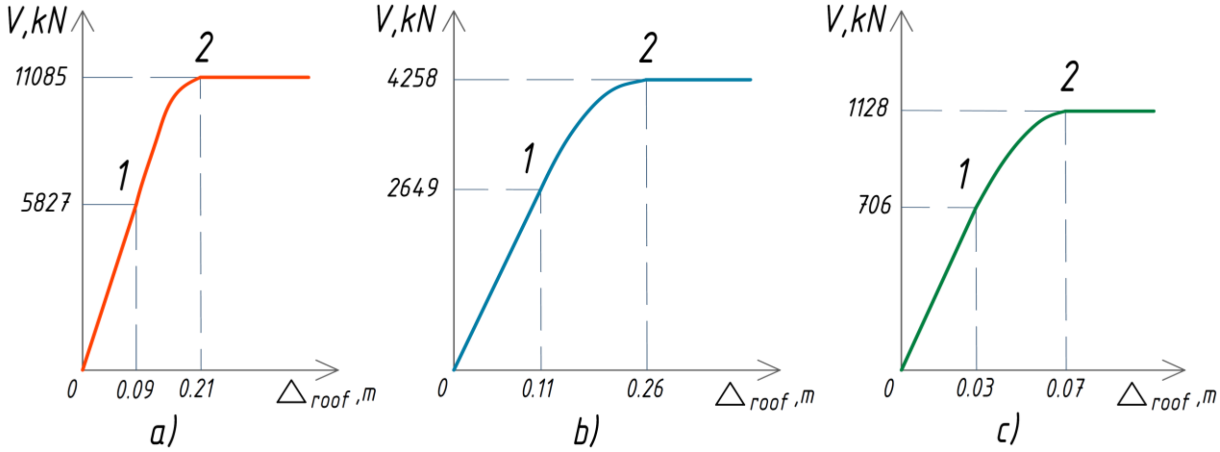

Figure 6. Curves describing load-bearing capacity for buildings
a – S_{MS} with maximum reinforced load-bearing framing;
b – S_{PS} with partially-reinforced load-bearing framing;
c – S_{tip} typical

E. Estimations of the damages for each seismic impact

It is necessary to set source data to estimate damages and solve an economic problem:

1. Life cycle of a building T is equal to 50 years.
2. Within these life cycle timings there will be a number of earthquakes with different intensity

Earthquake intensity	Number of earthquakes	Average annual number of earthquakes $\bar{L}$
Rate 6	5	0.10
Rate 7	4	0.08
Rate 8	3	0.06
Rate 9	1	0.02

To estimate the damages in each frame type caused by each earthquake specified there is computation based on time history analysis using the software Nonlin taking the range of rate 9, 8, 7 and 6 impacts, and the maximum displacement is determined, which allows estimating the value of damages once it is plotted on the relevant curve describing load-bearing capacity. Thus, there is an opportunity to represent curves describing load-bearing capacity in a form of the ratio “horizontal base shear – roof displacement /value of real structural damage". The value of structural damage emerged within the 0 – 1 spacing of the curve is equal to 0 since buildings behave elastically and there no damages in structural members. As the point 2 corresponds to the failure collapse of the columns due to an ultimate bending moment then from the financial point of view it represents total costs of all 18 load-bearing columns К1 and К1-1 with due account for works associated with disassembly of damaged ones and mounting new ones on their spots. These data can be obtained from cost estimates for each type of a building. In the financial curves obtained by distance values based on time history analyses it is possible to evaluate the damage state in the fractions of building costs per each impact. Estimation results for each type of buildings are given in corresponding tables.

Table 1. Time history analysis results for Frame S_{tip} (typical)

Earthquake intensity	Accelerogram	Acceleration max/min, g	Displacement, m	Real Damage D_{rel}
Rate 6	IMPERIAL VALLEY 10/15/79 2319, BONDS CORNER, 140 (USGS STATION 5054)	0.050 -0.084	0.009	0
	KOBE 01/16/95 2046, SHIN-OSAKA, UP (CUE)	0.059 -0.042	0.013	0
	KOCAELI 08/17/99, ARCELIK, DWN (KOERI)	0.086 -0.084	0.009	0
	IMPERIAL VALLEY 10/15/79 2319, BONDS CORNER, 230	0.100 -0.087	0.015	0
	IMPERIAL VALLEY 10/15/79 2319, BONDS CORNER, UP (USGS STATION 5054)	0.052 -0.042	0.015	0
Rate 7	LANDERS 06/28/92 1158, YERMO FIRE STATION, 270 (CDMG STATION 22074)	0.169 -0.245	0.042	0.0045
	KOCAELI 08/17/99, IZMIT, 180 (ERD)	0.147 -0.152	0.036	0.0023
	CHI-CHI 09/20/99, TCU045, V	0.181 -0.361	0.036	0.0023
	SUPERSTITION HILLS 11/24/87 13:16, EL CENTRO IMP CO CENTER, 090 (CDMG STATION 01	0.136 -0.258	0.031	0.0004
Rate 8	FRIULI, ITALY 05/06/76 2000, TOLMEZZO, 270	0.299 -0.315	0.083	1
	DUZCE 11/12/99, DUZCE, 180 (ERD)	0.307 -0.348	0.071	1
	CHI-CHI 09/20/99, TCU065, N	0.362 -0.603	0.092	1
Rate 9	CAPE MENDOCINO 04/25/92 1806, RIO DELL OVERPASS FF, 360 (CDMG STATION 89324)	0.549 -0.479	0.140	1

Based on the data from this Table we can deduce that when there is the rate 6 earthquake for this type of frame there are no structural damages. When there is the rate 7 earthquake the columns undergo to inelastic deformations and few of them are subject to be replaced. When there are the rate 8 and the rate 9 earthquake displacements exceed ultimate ones, and the building is forecasted to fail. Replacement costs for all the columns of Frame S_{tip} in the fractions from total building costs amounts to 0.015 in accordance with cost estimations.

Table 2. Time history analysis results for Frame S_{PS}

Earthquake intensity	Accelerogram	Acceleration max/min, g	Displacement, m	Real Damage D_{rel}
Rate 6	IMPERIAL VALLEY 10/15/79 2319, BONDS CORNER, 140 (USGS STATION 5054)	0.050 -0.084	0.007	0
	KOBE 01/16/95 2046, SHIN-OSAKA, UP (CUE)	0.059 -0.042	0.007	0
	KOCAELI 08/17/99, ARCELIK, DWN (KOERI)	0.086 -0.084	0.007	0
	IMPERIAL VALLEY 10/15/79 2319, BONDS CORNER, 230	0.100 -0.087	0.008	0
	IMPERIAL VALLEY 10/15/79 2319, BONDS CORNER, UP (USGS STATION 5054)	0.052 -0.042	0.001	0
Rate 7	LANDERS 06/28/92 1158, YERMO FIRE STATION, 270 (CDMG STATION 22074)	0.169 -0.245	0.034	0
	KOCAELI 08/17/99, IZMIT, 180 (ERD)	0.147 -0.152	0.028	0
	CHI-CHI 09/20/99, TCU045, V	0.181 -0.361	0.026	0
	SUPERSTITION HILLS 11/24/87 13:16, EL CENTRO IMP CO CENTER, 090 (CDMG STATION 01	0.138 -0.258	0.036	0
Rate 8	FRIULI, ITALY 05/06/76 2000, TOLMEZZO, 270	0.299 -0.315	0.064	0
	DUZCE 11/12/99, DUZCE, 180 (ERD)	0.307 -0.348	0.069	0
	LOMA PRIETA 10/18/89 00:05, CAPITOLA, 090 (CDMG STATION 47125)	0.368 -0.443	0.054	0
Rate 9	CAPE MENDOCINO 04/25/92 1806, RIO DELL OVERPASS FF, 360 (CDMG STATION 89324)	0.549 -0.479	0.140	0.004

From the time history analysis results for Frame S_{MS} it is clearly seen that there are no structural damages for all impacts except the rate 9 earthquake. When there is rate 9 earthquake the columns undergo to inelastic deformations and few of them are subject to be replaced. Replacement costs for all the main columns K1 and K1-1 of Frame S_{PS} in the fractions from total building costs amounts to 0.018 in accordance with cost estimations.

Table 3. Time history analysis results Frame S_{MS}

Earthquake intensity	Accelerogram	Acceleration max/min, g	Displacement, m	Real Damage D_{rel}
Rate 6	IMPERIAL VALLEY 10/15/79 2319, BONDS CORNER, 140 (USGS STATION 5054)	0.050 -0.084	0.007	0
	KOBE 01/16/95 2046, SHIN-OSAKA, UP (CUE)	0.059 -0.042	0.008	0
	KOCAELI 08/17/99, ARCELIK, DWN (KOERI)	0.086 -0.084	0.008	0
	IMPERIAL VALLEY 10/15/79 2319, BONDS CORNER, 230	0.100 -0.087	0.009	0
	IMPERIAL VALLEY 10/15/79 2319, BONDS CORNER, UP (USGS STATION 5054)	0.052 -0.042	0.001	0
Rate 7	LANDERS 06/28/92 1158, YERMO FIRE STATION, 270 (CDMG STATION 22074)	0.169 -0.245	0.035	0
	KOCAELI 08/17/99, IZMIT, 180 (ERD)	0.147 -0.152	0.028	0
	CHI-CHI 09/20/99, TCU045, V	0.181 -0.361	0.028	0
	SUPERSTITION HILLS 11/24/87 13:16, EL CENTRO IMP CO CENTER, 090 (CDMG STATION 01	0.138 -0.258	0.036	0
Rate 8	FRIULI, ITALY 05/06/76 2000, TOLMEZZO, 270	0.299 -0.315	0.065	0
	DUZCE 11/12/99, DUZCE, 180 (ERD)	0.307 -0.348	0.070	0
	CHI-CHI 09/20/99, TCU067, N	0.325 -0.302	0.071	0
Rate 9	CAPE MENDOCINO 04/25/92 1806, RIO DELL OVERPASS FF, 360 (CDMG STATION 89324)	0.549 -0.479	0.085	0

Estimation time history analysis results for Frame S_{MS} show no structural damages and the building behaves elastically for all the earthquake intensities specified in the Table 3. So we may deduce that there is no necessity to carry out repair works after earthquakes have been emerged.

Cost plots are given in Figure 7a-c. Repair costs after each earthquakes (i.e. damage state) can be determined doing interpolation by the value of displacement obtained from time history analysis estimations in accordance with corresponding intensity. Having all the damage state values obtained by the values of displacement corresponding to different impacts (rate 6, 7, 8 and 9) and given on the Tables1-3, we may evaluate cost-effectiveness for each retrofit scheme and determine strengths and drawbacks considering different life-cycle timings for different objects. Thereat, it is assumed that maximum structural roof displacements, which exceed ultimate values, can cause a collapse and the damage will amount the total cost of the building equal to 1.

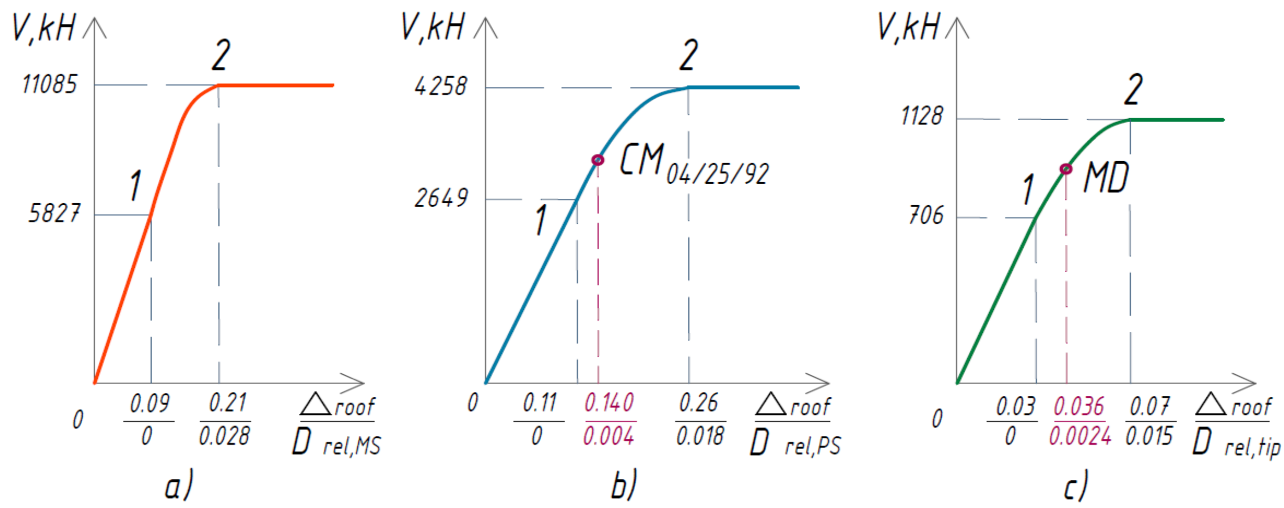

Figure 7. Financial curves for frames
a – S_{MS} with maximum-reinforced;
b – S_{PS} partially reinforced (the point $CM_{04/25/92}$ corresponds to rate 9 earthquake CAPE MENDOCINO 04/25/92 1806, RIO DELL OVERPASS FF, 360 (CDMG STATION 89324) (see Table 2), where the displacement specified and the corresponding damages are obtained);
c – S_{tip} typical (the point *MD* corresponds to the average displacement value specified on the basis of four rate 7 impacts (see Table 1) and the corresponding damage)

F. Computation of cost-effectiveness due to anti-seismic measures

With the values of the damage state for each building due to each seismic impact there is an opportunity to compute the cost-effectiveness E with the help of the Eq (2). Whereas the first term can be eliminated as it accounts for the income profit. In this case, the most beneficial option is selected with regard to the value size of costs associated with seismic retrofit, and the difference $D - D_{pr}$ can be represented as D_{rel}. Then the equation for the cost-effectiveness E will be as follows:

$$E=-K_{ant}\text{-}f(k,N)\sum_{I=I_{min}}^{I=I_{max}} L(I)\cdot D_{rel}, \qquad (4)$$

The value E is estimated for each frame at seven points which specify a certain period of the building life-cycle. Results are presented as plots depicting the ratio "cost-effectiveness E – building life-cycle N", a peculiar curve corresponds to each frame. When computing the parameter $f(k, N)$ the variable representing the profitability is considered equal to d=0.1, and d^*=0.03, but the value may change depending on the value of the profit.

Results and Discussion

G. Analysis of the results obtained

It is clearly seen from the plot in Figure 8 that the curves for frames S_{PS} and S_{MS} which correspond to the value E are parallel. It can be explained by the fact that when computing the value E for the building S_{PS} the only structural damage is considered, which is equal to 0.004 from total costs of the considered building. However, as a matter of fact, after an earthquake has been emerged there will be the damages associated with both structural members and non-structural ones (partition walls, suspended ceiling and etc.), members of utility services, engineering and manufacturing equipment, and site improvements can be damaged as well. As it is seen from the capacity curve (Figure 6b) and financial curve (Figure 7b) created for Frame S_{PS}, the damage state characterized by the displacement obtained from the computation made for the rate 9 time history, are beyond the level of standard "Operational" performance, which indicates the necessity to terminate operation of manufacturing until the damages are fixed. Termination incurs losses due to production downtime and loss of revenue, which estimation methods are suggested in the works [2, 8, 9, 10]. If we do an approximation and increase the structural damage value, for example, twice, then it is seen from the plot in Figure 9 that the cost-effectiveness curves for the frames S_{PS} and S_{MS} by the end of 50-years life-cycle period considered coming close to each other. Nevertheless, from investor point of view, it is economically feasible to execute partial reinforcement, since repair costs may not realized. It should be noted that the cost-effectiveness of the Frame S_{tip} with account of all types of damages, is dramatically going down with the time passed.

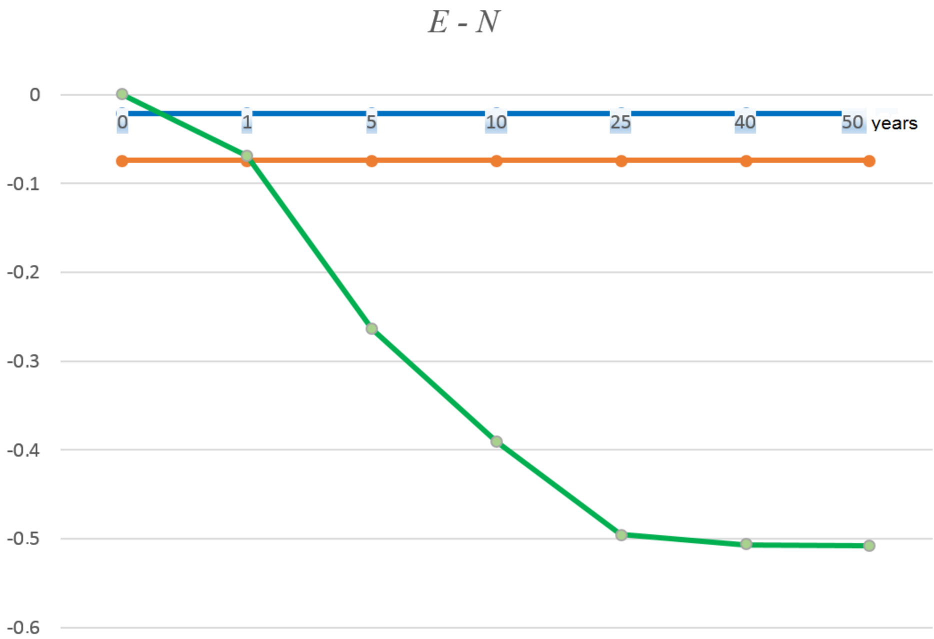

Figure 8. Dependence Plot
“cost-effectiveness E – life-cycle of the building N” considering only structural damage
– S_{PS} partially reinforced; – S_{MS} maximum reinforced; – S_{tip} typical

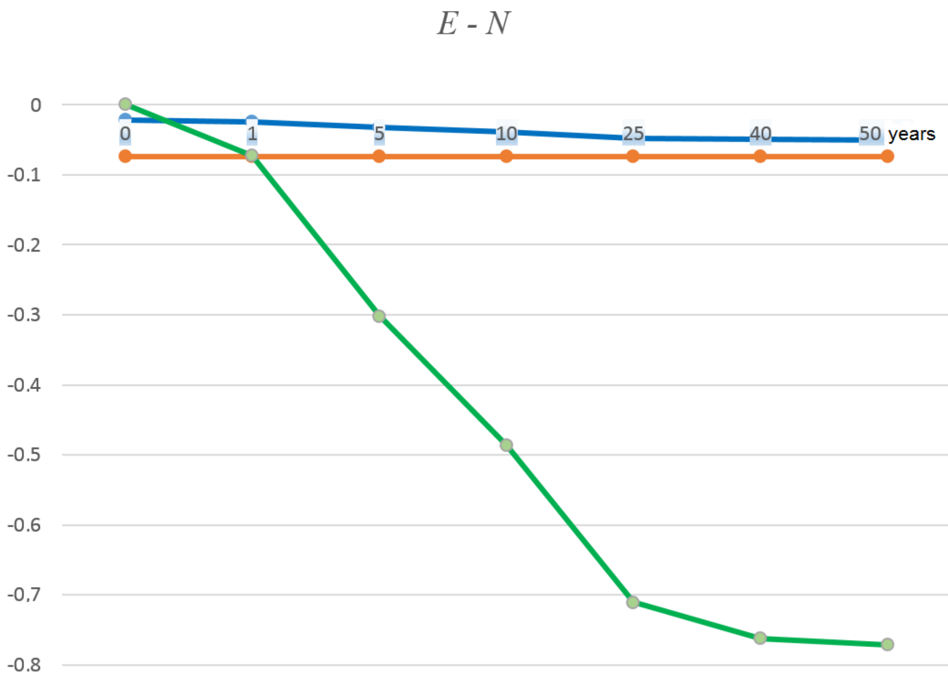

Figure 9. Dependence Plot
“cost-effectiveness E – life-cycle of the building N” considering total damage
– S_{PS} partially reinforced; – S_{MS} maximum reinforced; – S_{tip} typical

Thus, the higher cost-effectiveness E over the whole period can be achieved, as it seen, when designing the building with partially-reinforced frame. The cost-effectiveness value depending on the structural strengthening can be determined using procedure proposed in this study. At the same time, maintenance of Frame S_{tip} implies risks associated with high cost of repairs after each rate 7 earthquake or complete recovery after rate 8 and 9 earthquakes.

Conclusions

The method of comparative economic analysis for the range traditional seismic retrofit schemes of frames is developed in the article. The method provides the economic effect evaluation algorithm of applying a certain seismic retrofit scheme for the building designed on areas of seismic hazard.

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