Strength-strain state of the structures with consideration of the technical condition and changes in intensity of seismic loads

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Abstract. The paper focuses on determining the reserve in load bearing capacity of the building structures reconstructed in the seismic zone of Ukraine. The authors propose the method for simulation of the earthquake load acting on the structure with account of nonlinear material properties. In design of buildings and structures in seismic zones, in addition to calculations for the main load combination, calculations for the emergency combination of loads should also be made with account of the following seismic impact - minor earthquake (ME), design earthquake (DE) and max design earthquake (MDE). To evaluate behaviour of structures in earthquake load beyond elasticity, the nonlinear pushover analysis is used. With this the technique it is possible to consider the nonlinear properties of structures based on their bearing capacity (safety margins) rather than using the coefficients of inertial forces, as in the spectral method. The nonlinear static analysis of building structures allows you to consider the higher mode shapes of vibrations, interaction in the system 'overground structure – foundation soil'.

1. Introduction
Reliability and safety of buildings and structures tend to be associated with scientific approaches to simulation the actual behaviour of structures for normal and emergency cases. An important issue is the option to check the process of deformation and accumulation of damages by materials of structures over time, and, as a result, changes in behaviour of the structural system as a whole, as well as the possible destruction of structural components, so the structure is in emergency condition and may collapse. If the changes in the stress-strain state of the structure are considered in design practice at all stages of the life cycle, then it is possible to provide reliable estimation of the stress-strain state even during design process and carry out multi-variant numerical experiments.

2. Problem statement
This study aims to solve the problem of structural safety of buildings located in a seismic zone and subjected to reconstruction. This problem is solved based on a set of scientifically based methods for numerical simulation of the stress-strain state of structures with account of the categories of technical condition and analysis methods for structures with account of nonlinear deformation [1]. Numerical simulation of structures in the maintenance stage allows us to identify the reserve of the bearing capacity of structures that is necessary during reconstruction.
Scientific novelty of the results is in the fact that the authors obtained new scientifically based results aimed at creating methods for simulation and analysis of high-rise buildings at maintenance stage taking into account the categories of technical condition and changes in seismic load. The results of numerical simulation were compared with the results of the technical survey of the high-rise building.

3. Problem solution
In design of buildings and structures in seismic zones, in addition to calculations for the main load combination, calculations for the emergency combination of loads should also be made with account of the following seismic impact - minor earthquake (ME), design earthquake (DE) and max design earthquake (MDE). Seismic loads corresponding to the ME may be used in the design of buildings and structures of CC1 consequence class using detailed maps of the ZSR-2004-A0 (for the territories of the Autonomous Republic of Crimea and the Odessa region).

However, the earthquake load, even within the construction site, may differ from the accepted area by 1-2 points depending on the geological and hydrogeological conditions of the site. It causes an increase in seismic loads on the building by 2-4 times. Therefore, to clarify the seismicity, the construction site is divided into microzones [2, 3].

Seismic impact is the most dangerous type of dynamic load. It may cause significant structural damage, and in some cases lead to partial or complete destruction of buildings [4].

In earthquake analysis of buildings, two main analysis methods are used: the spectral method (engineering) that includes decomposition by mode shapes and direct dynamic (direct integration over time). Modern building codes [5, 6] also suggest to use the method of simplified nonlinear static analysis of a building in earthquake - Pushover Analysis. Pushover is a static nonlinear analysis in which the vertically loaded design model of the structure undergoes a monotonic increase in horizontal seismic load with control of horizontal displacement.

With this technique it is possible to consider the nonlinear properties of structures based on their bearing capacity (safety margins) rather than using the coefficients of inertial forces, as in the spectral method. Nonlinear parameters of materials are considered due to redefining the integral stiffness properties for the finite elements of the model at each step of load application. Then the point of intersection of the bearing capacity curves is computed as well as the reaction spectrum (the point of dynamic equilibrium, based on which the behaviour of structure is determined). The load is applied step by step from the horizontal seismic forces until either 1) the specified skew of the floors is reached, or 2) the building is destroyed or 3) the maximum value of the forces is reached (if criteria 1 and 2 were not applied). The spectral acceleration-spectral displacement diagram is generated. When reduced normative diagram of the spectrum of reactions is superimposed onto the spectrum diagram of the bearing capacity, the state point of design model is determined.

The universal pushover analysis algorithm is implemented in LIRA-SAPR program and may be applied to all possible reaction spectrums. These are the dependences 'dynamic coefficient $\beta$ - period of vibrations $T$ and 'acceleration $Sa$ - displacement $Sd$'. It is also possible to apply arbitrary user-defined reaction spectrums as well as reaction spectrums computed from user accelerograms. To simplify the procedure, horizontal forces are recommended to be distributed according to the main mode shape of the building. Using iterative procedures, the max seismic displacement of nonlinear system with one DOF is determined by an equivalent linear system (the period and damping coefficient of which is greater than the initial values for the nonlinear system). Using this method, the 'capability' curve must be transformed into the 'spectral acceleration-displacement' dependence curve; then the 'capability' curve and the seismic 'requirement' are located in the same coordinate system.

Algorithm is the following:
1. A multi-mass design model of the building is generated. Then the earthquake analysis is carried out in linear statement. The following data is determined: concentrated masses at each level along the height; frequencies and periods of natural vibrations; the ordinates of mode shapes of natural vibrations; inertial forces at each level along the height; areas of reinforcement for RC structures are selected.
2. From all the computed mode shapes of natural vibrations, the shape with the greatest modal contribution is selected, because the reaction of structures or the 'bearing capacity' curve depends on the selected distribution of horizontal forces.

3. Inertial forces from the selected component are transferred to a separate load. Characteristic load is applied to the model in order to recalculate the integral stiffness parameters of the inelastic model and determine the initial stress-strain state.

4. Then linear design model is transformed into physically nonlinear one, where its load history is generated. The load history includes in sequence: full vertical load; horizontal earthquake loads applied step-by-step (they correspond to mode shape of natural vibrations with the greatest modal mass).

   The number of load histories is not limited. Within the one design model, it is possible to consider different options for the application of earthquake load.

5. At each step of increasing the inertial load of the multi-mass design model, based on the generalized spectral displacements $S_d$ and generalized spectral accelerations $S_a$, a nonlinear bearing spectrum is generated for the multi-mass design model with account of the nonlinear material properties:

\[
S_{aj} = \frac{\sum_{i=1}^{n} m_i d_{ij}^2}{\left(\sum_{i=1}^{n} m_i d_{ij}\right) S_{ij}},
\]

where $m_i$ is the mass concentrated in the $i$-th level (storey) of design model, $d_i$ is the horizontal displacement of the $i$-th storey of design model under inertial loads $S_i$ by the $j$-th mode shape.

\[
S_{dj} = \frac{\sum_{i=1}^{n} m_i d_{ij}^2}{\sum_{i=1}^{n} S_{ij} d_{ij}} S_{aj}.
\]

6. As a result, two generalized diagrams are obtained: the first is the acceleration spectrum (the diagram in coordinates 'spectral acceleration $S_a$ – spectral displacement $S_d$'); the second is the bearing capacity spectrum (diagram in coordinates 'horizontal force $V$ - displacement $S_d$'). The diagram is obtained automatically by multiplying the obtained spectral accelerations by the modal mass.

7. The diagram 'horizontal force $V$ - displacement $S_d$' (Figure 1) is converted to bilinear view based on the equal areas (energies $E$) of the nonlinear and bilinear diagrams. With the idealized bilinear diagram, the compliance coefficient for the model is found. With this coefficient, the standard (or given) reaction spectrum will be reduced for the subsequent computation:

\[
\mu = \frac{d_m}{d_r},
\]

where $d_m$ is the max displacement of the multi-mass design model according to the results of nonlinear static analysis, $d_r$ is the displacement of the multi-mass design model corresponding to the yield stress of the multi-mass design model.

Figure 1. The ‘horizontal load – displacement’ diagram.
8. For the obtained compliance coefficient $\mu$ ($\mu = 1; 2; 4; 6$), diagram of normative dependence 'spectral acceleration $S_a$ – spectral displacement $S_d$' is elected, it is also called the seismic 'requirement' diagram. For intermediate values of $\mu$, the diagram is generated by interpolation.

9. Then, the 'bearing capacity' curve - the diagram $S_a (S_d)$ obtained as a result of nonlinear analysis and the seismic 'requirement' is located in the same coordinate system.

10. The intersection point of the normative and nonlinear diagrams is called the state point: $dS$ is the required generalized nonlinear displacement at which the stress-strain state of the whole structure is determined; $aS$ is the corresponding acceleration at which the stress-strain state of the whole structure is determined. Displacement $dS$ - is the required generalized nonlinear displacement, for which the stress-strain state of the whole structure is determined.

11. Horizontal force of the state $V_S = aS \times M$. Based on the $dS$ value, the processor corrects the step of nonlinear analysis in the area of intersection with the diagram of the seismic requirement. So, required stress-strain state of the whole model is obtained.

![Diagram 1](image1.png)  
![Diagram 2](image2.png)

**Figure 2.** Bearing capacity spectrum (diagram 1) superimposed onto acceleration spectrum (diagram 2).

In the output data of the pushover analysis, as for any physically nonlinear model, it is possible to view the resulting displacements within the load history (output data may be displayed for every step of load application), evaluate the forces in elements of the model, evaluate the state of materials, i.e. get information about the destroyed elements. You could also generate the diagram for 'bearing capacity spectrum'. Analysis of buildings according to [5] includes analysis with 1 method of reduction.

The final phase is to find the intersection point of the reduced reaction spectrum with the spectrum of bearing capacity. This point is called a state point. The stress-strain state of the whole system is determined for this point. The bearing capacity of the structure in earthquake load is also evaluated for this point.

4. Example
The building of the hotel complex was built in 1999 - 2001 at the New pier of the Odessa seaport; the pier was made up in 1968. The hotel building is directly adjacent to the Marine terminal station. The hotel was put into service in 2001. From 2010 up to the present time, the building is out of service with the min necessary conservation work and maintenance of its life.

The building of the hotel complex consists of two separate parts separated by deformation seams:
- two-storey stylobate part on the cover of which people and cars are moved;
- high-rise central part, the total height of which is 19 floors ($h = 76.5m$).

Technical inspection of the building was conducted according to [5, 7] on the basis of:
- geotechnical survey;
- structural, architectural planning decision of the building and the sizes of its elements;
- visual and instrumental inspection of load-bearing and enclosing structures;
- photo fixation of defects and damages of structures;
- the percentage of physical deterioration and the category of technical condition of load-bearing structures and the building as a whole.

The spatial rigidity of the building is provided by a monolithic RC load-bearing frame that consists of vertical load-bearing elements - columns, pylons, diaphragms and horizontal disks of the inserted floors. As a result of a comprehensive survey of the hotel building, the strength, stability and physical depreciation of the building as a whole and its individual structural elements were checked. Defects and damages that arose during the design, construction and maintenance of the building were identified. The percentage of physical depreciation of structures and the category of technical condition of structural elements and the building as a whole are determined. Real defects that influence the structural safety of the building are shown in Figure 3.

With non-destructive methods, the strength of concrete and the actual reinforcement of load-bearing RC structures were determined. Technical inspection showed that the physical depreciation of RC structures is 15%; the load-bearing capacity, stability and rigidity of the load-bearing structural members of the building are adequate to take the maintenance loads. The load-bearing frame of the building is in satisfactory technical condition (category 2) and is suitable for further operation.

![Image](image.jpg)

**Figure 3.** Photo fixation of defects and damages:

a) Vertical crack caused by corrosion of reinforcement, peeling off the protective layer in the RC column of the first floor at the intersection of axes 'Ds, 31', el. +0,350;
b) Crack in the column at the intersection of the '31' axes between the 'Vc-Bc' axes;
c) Crack of deformation seam on axis 'A' in axes '6'-7" at the el. +0,350;
d) Crack of the deformation joint between the axes 'He-B / Ge" at el. +0,350.

The hotel is located in the seismic zone. Verification analysis was carried on dead, temporary, vertical snow load, horizontal static wind load and earthquake load. According to Table 2.4 of the [8], the coefficient of responsibility for the building is 1.2.

According to SNIP II-7-81, which was in force during the design of the building, Odessa belonged to the territories with seismicity of 6 units of magnitude. But taking into account the soil conditions of
the site - water-saturated soils with high liquidity indexes, the building is analysed on 7 units of magnitude. Class of consequences (responsibility) of the building according to DSTU-N B B.1.2-16: 2013 - CC3 [8, 9].

According to [5, 10], the effective seismicity for buildings of high responsibility in Odessa (see the map of the ZSR-2004 C) is 8 units of magnitude. Given the soil properties of construction site (soil category III), the building of a high-rise hotel should take an earthquake load of 9 units of magnitude. Earthquake load is included in emergency cases and is taken into account in analysis of structures in ultimate limit state.

On the basis of the project documentation, a 3D computer model of the building was generated. The model has a vertical Z-axis and two mutually perpendicular axes X, Y. General view of design model is presented in Figure 4.

Design model of the building consists of the basic model with elements of the frame (pylons, walls of rigidity diaphragms, walls of technical floors) and floor slabs, beams and roofs. The base is a slab with thickness 500 mm on the soil base under the 2-floor part separated by a deformation seam. Solid slab pile grillage with thickness 700 mm under the 4-storey and 1200 mm below the high-rise parts. The number of piles is 450 pcs, based on 225t of bearing capacity. The bearing capacity of the pile is determined by the static test method.

![Figure 4. Design model of the building (general view).](image)

The analysis was carried out on combinations of different loads, specific combinations of design loads were considered as well as the main combinations.

Max horizontal displacements occur in vertical wind load and are equal to 20.9 mm. According to [11], allowed displacements of the building are \( f \leq H/500 \), where \( H \) is conditional height of the building, it is equal to 72.5 m; \( 20.9 \text{ mm} \leq 72500/500 = 145.0 \text{ mm} \). The condition is fulfilled.

Ultimate settlements of the building according to [3] - 10.0 cm. Max settlements of the base slab according to analysis - 47.9 mm <100 mm - the condition is fulfilled. The ultimate relative difference of settlements according to - \( \Delta/L = 0.002; \Delta = 47.9-18.8 = 29.1 \text{ mm}; L = 23200 \text{ mm}, \text{ from the length of the stylobate part}; \Delta/L = 29.1/23200.0 = 0.001 <0.002 - the condition is fulfilled.

Ultimate relative difference of strain in floor slab at elev. +3,270 - \( \Delta/L = 0.002; \Delta = 50.5-20.4 = 30.1 \text{ mm}; L = 18600 \text{ mm}, \text{ length of building}; \Delta / L = 30.1/18600.0 = 0.0016 <0.0020 - the condition is fulfilled.
Forces in piles do not exceed the bearing capacity of the pile 225.0 t. By analysis, \( F_{\text{max}} = 152.0 \) t <225.0 t - the condition is fulfilled.

In order to comply with the requirements of the earthquake analysis, it was necessary to consider all mode shapes that make a significant contribution to the total response of the building, provided that the sum of the effective modal masses of considered mode shapes is 90% of the total mass of the building, or at least 85% for vibrations of the building in the horizontal direction [5, 12].

Since to evaluate and restore the earthquake resistance of buildings that are in operation and taking account of their actual technical condition (cracks, defects, damage), a non-linear static analysis was carried out according to the recommendations of DBN 2014. Also, to evaluate behaviour of structures in earthquake load beyond the elastic limits, the nonlinear static pushover analysis was carried out in LIRA-SAPR program [13] (Figure 5).

![Figure 5. Dialog box to define parameters for pushover analysis.](image)

For a nonlinear problem, load history is generated in the Modeling nonlinear load cases dialog box. In the Analysis method list, select the pushover option.

On the Parameters tab, define the following data:

- No. of dynamic load case (earthquake load case, except modules 38, 46);
- No. of load case with inertial forces;
- coefficient for inertial forces.

If the Max allowable skew option is selected and its value is defined, then you should specify pairs of nodes for which the skew will be computed during analysis procedure.

If the Max coefficient for inertial forces option is selected, then analysis will continue until the specified value is exceeded.

The Step of load application is selected from the list: automatic (the step is selected automatically during analysis) or defined (define the step value in the appropriate box).

Then, in the History area, it is necessary to expand the history and define data for all load cases preceding (and only previous) the earthquake one.

Thus, a story will be generated; it consistently includes:
- loads preceding inertial forces (as a rule, vertical load);
- step-by-step applied horizontal earthquake loads corresponding to the mode shape of natural vibrations with the greatest modal mass.

Based on the specified max skews of the floors, the earthquake load increases. Upon reaching the max skews, the inertial forces are not increased more.

**Figure 6.** Displacements of the building along the X and Y-axes.

Nonlinear static analysis is the first phase in determining the spectrum of bearing capacity of a multi-mass design model. The bearing capacity spectrum is the ratio of the base shear in earthquake load to the horizontal reaction (displacement) of the building. As a result of a nonlinear static analysis, the displacements of each level at each loading step were computed (Figure 6).

**Figure 7.** Ultimate skews of storeys at selected nodes.
For the obtained displacements, skews are determined at different storeys of the multi-mass design model. The strength of the building structure as well as compliance of the computed skews with allowed values is checked. The results of certain skews for each storey in design model are presented in Figure 7.

The bearing capacity spectrum of the multi-mass design model for the $j$-th mode shape is generated using dependences (1), (2). The compliance coefficient $\mu$ is determined by formula (3). Effective coefficient of inertial forces (reduction coefficient) is a decrease in the earthquake reaction due to nonlinear properties; it depends on the elastic reaction and its corresponding displacement during vibrations of the building in earthquake load.

5. Conclusions
According to the results of modal analysis, 140 mode shapes are considered in horizontal earthquake load along the X-axis - the sum of modal masses is 88.3%; along the Y-axis - 210 mode shapes, 82.8%; in vertical load - 19 mode shapes with 91.7% of the sum of modal masses.

After nonlinear static pushover analysis, the research engineer could evaluate the real reserves of the structure by taking into account the plastic and other inelastic properties of the structures (Figure 8). The obtained compliance coefficient may be used in analysis by spectral method. When reduced normative diagram of the spectrum of reactions is superimposed onto the spectrum diagram of the bearing capacity, their intersection point (with coordinates $dS$, $aS$) is determined. This is the state point. For this point, the stress-strain state of the whole design model is determined and the possibilities of the structure to work beyond the elastic limit under earthquake load are evaluated.

For the selected structure, the max skew (0.03 m) between nodes 129371 and 134286 was with coefficient to inertial forces 0.28614. Based on this data, the spectral displacements $Sd$ (0.012 m) and spectral accelerations $Sa$ (0.754 m/s$^2$) were computed at the state point.

**Figure 8.** Spectrum of bearing capacity and normative spectrum of acceleration for the building.

Diagrams of the bearing capacity spectrum and the acceleration spectrum of the structure are presented in Figure 8.
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