Behavior of a multi-story building under seismic loads when taking into account the viscoplasticity of the soil base.

L'interaction entre les constructions du bâtiment sous charges sismiques tout en tenant compte de la viscoplasticité de la base du sol.

Boyko I.P., Sakharov O.S., Sakharov V.O.
KNUCA, Kyiv, Ukraine

ABSTRACT: This article presents an analysis of the dynamic behavior of the 3D “soil base – foundation – above-ground construction” system that was conducted via the automotive system of scientific research (ASSR) “VESNA”. In particular, the presented results show the effect of the use of elastic, visco-elastic and elastic-viscoplastic models on the behavior of the building under seismic loads while taking into account a seismic isolation system. Taking into consideration the non-linearity of the soil’s physical properties allowed the model to estimate the actual spatial position of the engineering structures after seismic action. The model demonstrated that the greatest forces within the piles appeared below the pile heads.

RÉSUMÉ: Cet article présente une analyse du comportement dynamique de la «base au sol - fondation - construction du bâtiment" en 3 dimensions qui a été menée par les moyens du logiciel de recherche scientifique (ASSR) "VESNA". En particulier, les résultats présentés montrent l'impact des modèles élastiques, visco-élastiques et élasto-viscoplastiques sur le comportement du bâtiment sous des charges sismiques tout en tenant compte d'un système d'isolation sismique. Tenant compte de la non-linéarité des propriétés physiques du sol, le modèle a permis d'estimer la position des constructions du bâtiment après l'action sismique. Le modèle a démontré que les plus grands efforts dans les pieux sont révélés en-dessous des têtes de pieux..

KEYWORDS: Numerical modeling, dynamic, seismic, visco-plastic model, soil base, pile foundation, high-rise building, VESNA

1 INTRODUCTION

Seismic loads often result in additional stress in the building which may exceed forces generated by the static and wind-related load or even cause a completely different force distribution picture. The dynamic stress-strain state of the “soil base - foundation - building” system is substantially different from the static one. Results of the interaction of the engineering constructions with the soil base are largely dependent on the characteristics of each element. This leads to the emergence of new zones with a maximum load (vs static loading conditions).
The difficulty of a dynamic load assessment is caused by the necessity of a detailed analysis of the dynamic behavior of all the elements of the system. Seismic loads usually have a wide range of frequencies, eliminating the possibility of predicting the possible stress state of the system during the load.

1.1 *Problem definition and the finite element model.*

Existing spectral methods do not provide a complete picture of the interaction between the solid base and engineering constructions. In addition, methods based on Vinkler’s model of the soil foundation base can not be considered satisfactory. In this case it is impossible to correctly take into account the inertial forces of the soil base. Foundations under tight pinching conditions lead to even larger errors due to the difficulty of precisely modeling interactions within the system.

The presented article contains an analysis of a real multi-story building, located on a landslide slope in the seismically-active area of the Crimean Republic. The building contains 16 floors with the lowest floor occupied by parking facilities. Load-bearing constructions consist of 220mm floor slab, an interior load-bearing column (with elevator shaft), diaphragm (up to the 5th floor) and columns. The ground (parking) floor uses a solid perimeter 400mm wall. The floor slab area steadily decreases with each floor from 900 m^2 to 300 m^2. Columns have a section 400x400 mm with the contour inclined in accordance with the generatrix.

The soil base is represented by a talus layer about 10-18 m deep with a shifted mudstone layer (about 3-5 m deep) and an argilite foundation below that.

The initial building design called for drilling piles ($\varnothing$620 мм L35 м), embedded into the argilite bedrock. Also, taking into account the high seismic activity (up to the XIII – IX degree by MSK-64) and overall height of the building, the design called for a grillage slab with a thickness of 2m.

3D modeling was performed using the automotive system of scientific research (ASSR) “VESNA”. The finite element model consists of multi-layer soil mass and engineering constructions (Fig. 1). The size of the soil base was selected to minimize the effect of boundary conditions on the construction elements. Selected dimensions included 200 x 180m area with 155m depth under foundation.

In order to reduce boundary effects caused by wave reflection, boundary conditions were supplemented with absorption conditions according to the Lysmer model (J.Lysmer, R.L. Kuhlemeyer, 1969).

Seismic load action was modeled using 3D accelerogram taken from the set of standard synthetic accelerograms (ДБН B.1.1-12-2006) while taking into account the lowest eigenfrequency of the construction.

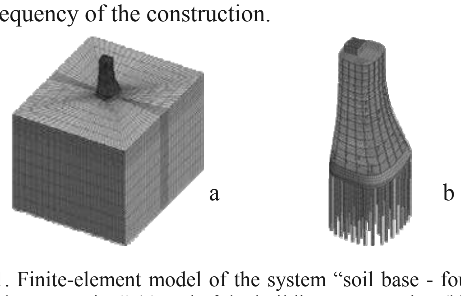


Figure 1. Finite-element model of the system “soil base - foundation – overhead construction” (a) and of the building construction (b) .

Construction eigenfrequencies and related forms were calculated (selected) while accounting for the possible stiff shift of the building in the absence of the boundary conditions. Lowest eigenfrequency period was determined to be T=0.56 s. Given the conditions above, normalized 30 second plot of the accelerogram No. 3 from the standard accelerogram set (ДБН В.1.1-12-2006) was used to model seismic activity (Fig. 2).

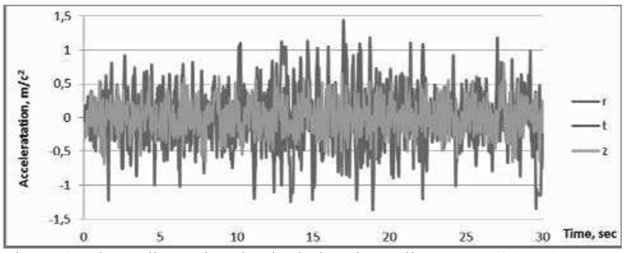


Figure 2. Three-dimensional calculational accellerogram (30 sec).

1.2 *Construction modelling under dynamic loads with elastic and viscoelastic components.*

In order to identify the physical processes that occur due to attenuated oscillations, the initial stage of the interaction analysis within the system “soil base - foundation - building” was performed using a linear model.

Under inertial forces due to the applied accelerogram, the building experienced significant oscillation both horizontally and vertically. Spatial movement analysis has shown that most movements occurred in the same direction as interial forces caused by the accelerogram (including building subsidence). Looking at graph of the absolute value of the displacement vector magnitudes (Fig. 3) it is clear that under load the building experiences an increase in oscillation amplitude. Such increase could be interpreted as a consequence of resonant processes within the system.

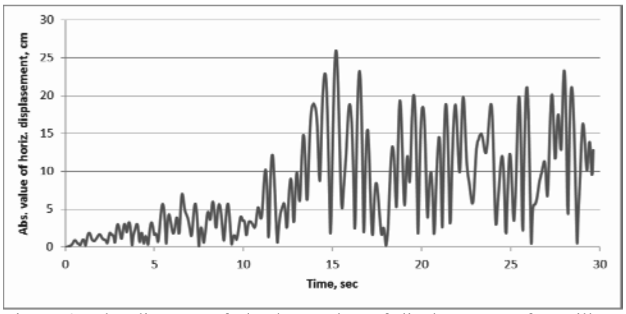
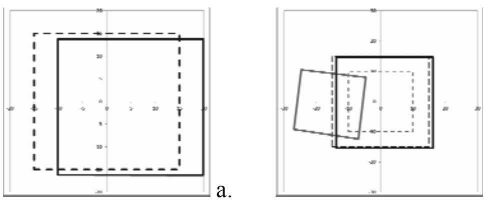


Figure 3. The diagram of absolute value of displacements for grillage slab in plane (elastic model without attenuation).

During load the maximum horizontal deviation of the grillage slab reached 26 cm at 15.1 seconds (Fig. 3), (Fig. 4, a).



a. b.

Figure 4. Maximal displacements in plane: a - of the grillage slab in the moment of time t=15.1 s (the coefficient of deformation increasing Kd=20); b – of the foundation and the covering plate t= 17.65 s, Kd=5)

Since the vertical load bearing structures consist mainly of the internal column (with elevator shaft) and outside columns, the building does not have high spatial rigidity. Due to the horizontal vibration and considerable height of the building, the upper floor experienced significantly higher oscillation magnitudes, reaching 341 cm and 297 cm at 17.65 s and 25.38 s respectively. Placing the rigidity core of elevator shaft near the building edges leads to twisting during oscillation (Fig. 4, b).

Taking into account that the oscillation of the whole system does not weaken over time, forces within the building would be at maximum - especially where resonance effect is manifested. Further analysis shows that, under dynamic load, the maximum forces are concentrated around zones with maximum rigidity - namely the core column with elevator shaft, diaphragm and outside columns. For this setting maximum forces start to appear after 15 seconds of load.

Vertical forces that appear in piles have alternating direction with compression forces reaching $20 \cdot 10^3$ kN and tensile forces reaching $19 \cdot 10^3$ kN. According to the design, the piles have sufficient length to transmit the load into the argilite soil layer, crossing the weaker soil.

Forces within the piles reach their maximum values at around 16 seconds, which correlates well with observations in the rest of the foundation. In reality, propagating waves will experience energy loss due to viscosity, plasticity or destruction of the medium. Thus, for a more precise assessment of the stress-strain state it is important to consider oscillation attenuation.

Currently there are several mathematical models that allow modeling of oscillation damping. Ukrainian design standards have only general recommendations with regard to the model selection. However, the American standard for nuclear power plant design (ASCE Standard 4-98, 1999) recommends to use one of the (four) specific models for taking into account seismic load - including Rayleigh’s two-parameter damping model. According to it, the attenuation matrix is a linear combination of mass and stiffness matrices, taken with the corresponding coefficients α and β (Eq.1)

$$[C] = \alpha[M] + \beta[K] \tag{1}$$

The implementation of this model relied on determining coefficients α and β through the damping parameters (ξ_{sum}, ξ_M, ξ_K), which are dependent on the selected natural frequencies ω_0, ω_1 and the logarithmic decrement oscillations δ (Eq. 2), (Eq. 3). Oscillation attenuation differs between soil and reinforced concrete engineering structures - thus each of mediums used different attenuation. Logarithmic decrement were set according to Ukrainian standard values which are δ=0.3 for the reinforced concrete engineering structures and δ=0.6 for soil.

$$\xi_{sum} = \frac{\delta}{\sqrt{4 \cdot \pi^2 + \delta^2}} = \xi_K + \xi_M \tag{2}$$

$$\xi_M = \frac{\alpha}{2\omega_0}, \qquad \xi_K = \frac{\beta \cdot \omega_1}{2} \tag{3}$$

Given that the calculation model relied on several attenuation parameters, Newmark’s direct integration method was chosen to solve the problem.

While the system load remained unchanged, calculations showed that accounting for attenuation process significantly affected not only the oscillation amplitude values, but the deformation characteristics of the whole structure. Movement amplitude decreased by a factor of 4, reaching 8.2 cm for grillage slab after 13.8 s of load (Fig. 5).

Vertical oscillation amplitude relative to the initial position did not exceed 4 cm.

Maximum moments arising in the grillage slab were also concentrated within other load bearing elements (interior load-bearing column, diaphragms and columns) during the time period from 10 s to 24 s.

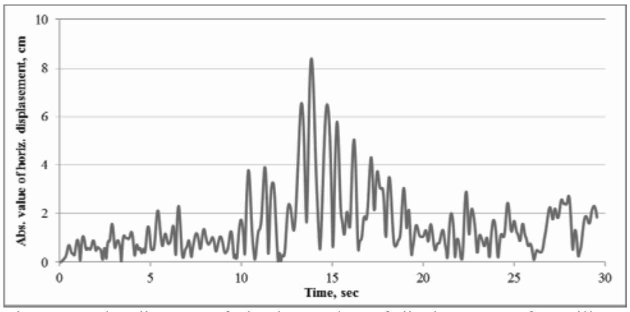


Figure 5. The diagram of absolute value of displacements for grillage slab in plane (visco-elastic model).

Forces in piles were reduced by a factor of ~3. The extreme forces were located at the pile head (maximum compressive reached $6.6{\cdot}10^3$ kN, tensile forces – $6.2{\cdot}10^3$ kN). But there are piles where the maximum force was below the head. For the majority of piles, the maximum forces were experienced during the 15 – 20 second time period.

Preliminary calculations showed that for the given seismic activity, oscillation magnitude for upper floors can reach around 1m, resulting in unacceptable operating conditions. In order to reduce seismic wave impact, use of a seismic isolation system has been advised. While such a system results in decreased overall building rigidity, it significantly reduces the resonance effect and helps to absorb oscillation energy by utilizing plastic forces within the damper. In order to implement the seismic isolation system, an additional damping layer was added into the grillage slab. Thus, the resulting foundation design consisted of a grillage slab, plate and damping layer. Grillage connected piles to the bottom plate. The plate was lying on top of the damping layer while being rigidly connected to the above-ground constructions.

Based on the results of previous studies, it was determined that increased local vertical stress values are primarily associated with the presence of a weak soil layer (with increased deformability). Conducted full-scale pile testing with shortened piles showed sufficient carrying capacity to accept building load. Taking into account that the building is located on the slope which is fixed with two retaining walls from both building's sides (slope strengthening structures) the piles' length was proposed to be decreased to 13 m not intersecting weak soil layer. The undertaken researches of piles' length impact on construction's stress condition corroborated this solution.

Dynamic problems that take into account visco-elastic plastic medium deformations while utilizing Newmark method will need large amount of iterations to calculate stiffness matrix. This prompts the use of direct explicit methods. In this work Wilkins' explicit method is used. One of the properties of the direct methods is a very small integration time step, resulting in a large number of iterations. However it allows the application of a visco-elastic plastic model for all mediums without significant algorithmic complexities or computational expenses. Such an approach permits the stress-strain mode (state) to be more exactly defined and research the interaction of the soil base- foundation- above-ground construction system's elements while taking into account such effects as detachment of structures from soil, consolidation of soil base etc. To describe non-linear soil's behaviour non-associative law of modified Mises - Schleicher - Botkin's criterion taking to account soil structural strength is used (Boyko, Sakharov 2004, 2005) (Fig. 6). Model parameters can be defined with known methods. Furthermore, soil properties measured on the actual site were used to identify correct parameter values for the model. Combined with dynamic viscosity with inherent parameters for different mediums (Rayleigh's model) this permitted to take into account the real processes of the soil base's deformation when interacting with engineering structures during complicated stressed mode and permits to project reliable and rational structures as a result.

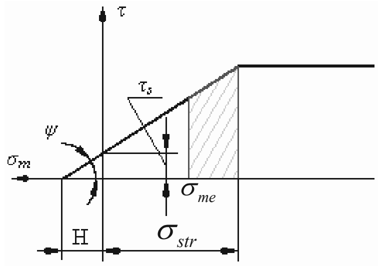


Figure 6. The modified criterion of Mises-Schleicher-Botkin with structural strength (σ_{me}, σ_{str} – the limit of elasticity and structural strength).

In this work, the traditional Mises - Schleicher - Botkin's model was supplemented with the condition of pre-boundary plasticity and the effect of unloading for hydrodynamical tensions.

Taking into account the unloading effect has significant importance for solving problems with cyclical loading. The realization presupposes than on the initial stage of loading up to structural strength, the value of which has been taken to be 20% more than natural hydrodynamic tensions (taking into account the building), the soil is deformed with a modulus of elasticity, the value of which is accepted from the results of labaratory research to be $E_{el}=5{\cdot}E_{def}$. At further load the soil is deformed with module of deformation, which is why the shifts are increasing. In the step of downloading the soil is deformed with the module of downloading. Within this problem this module was accepted as E_{el}.

The shown model allows the consideration of the prelimit plasticity, the visco-elastoplastic soil's work and the damper's material and also to estimate the position of the building after the end of the seismic load.

The specification of the work of soil and damper led to an increase of shifts in the plane of the building. Maximal amplitude increased from 8 to 10 sm. At the maximal amplitude the shifts are oriented mainly along the action of the radial component of the seismic load. At such conditions the oscillations in the horizontal plane were close to the neutral situation, which is why the building does not have tendencies to horizontal shifts. The consideration of the plastic work of the damping layer allows the calculation of the amplitude decreasing of the oscillations of the top floors of the building. These shifts differ significantly in character relative to the previous calculations without the damping layer, and their maximum value decreased 40% and became approximately 64 cm in the period of time from 15.1 to 24.5 seconds (Fig. 7).

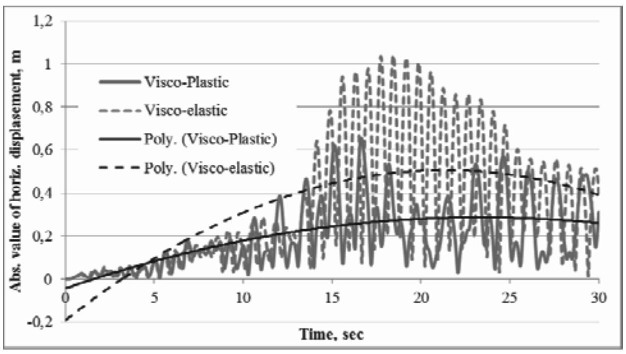


Figure 7. The diagram of absolute value of displacements for upper foundation slab (visco-elastoplastic model) and grillage slab (visco-elastic model)

The process of irreversible building settlement was fixed because of the consideration of the structure strength and the effect of download. The soil compaction occurs only during excess of stress of the structure strength. At irreversible soil compaction the deformation is with the deformation module, which corresponds to the real soil's work. As it can be seen from the diagram (Fig. 8), the process was developed to 20 s of load, after which the settlement became stable and at least exceeded the value of ~13 cm.

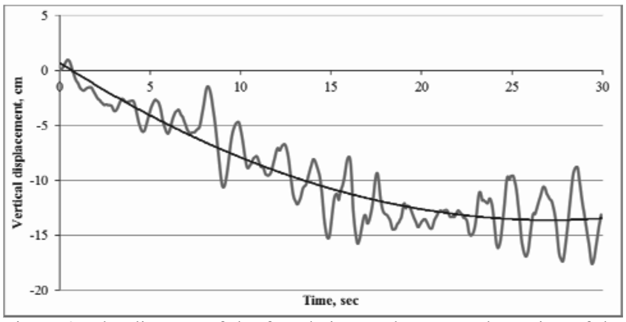


Figure 8. The diagram of the foundation settlement at the action of the seismic load

The stress state in the foundation constructions was decreased. At the top plate the moments became mostly 500-800 kN·m/m, but in the areas of stress concentrations in the zones of the elevator shaft, diaphragms and columns the local values of moments were near $4.5{\cdot}10^3$ kN·m/m. In grillage slab – to $3.3{\cdot}10^3$ kN·m/m.

At visco- elastoplastic formulation, the compressive forces in piles decreased ~40 % and changed from $1.7{\cdot}10^3$ kN to $4{\cdot}10^3$ kN at the head level. During stretching, the maximal efforts were from $0.5{\cdot}10^3$ kN to $3.5{\cdot}10^3$ kN in various zones. Notice that in this variant the piles are in homogeneous soil. But, despite of this, the significant stretching efforts have maximum tensile values below the head of the pile (Fig. 9).

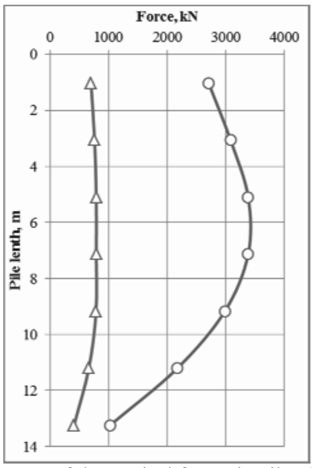


Figure 9. The diagrams of the vertical forces in piles (Δ – central zone, ο – contur zone)

2 CONCLUSIONS

It has been shown that while solving problems of seismic load influences on buildings and other structures it is necessary to consider the inertial mass forces of the soil base and the corresponding oscillation attenuation processes in the soil and constructions.

It has been found that under the influence of seismic loads, predominantly from inertial forces in the soil, zones of significant tensile forces can appear in the piles. These zones are located below the pile heads and must be taken into account when designing grillage for the structure.

It has been determined that the utilization of piles during seismic loads in layered soil bases with various deformation properties leads to the appearance of forces within these piles that can exceed the forces at the pile heads by as much as a factor of two.

The maximum forces within the structure can occur at various times and do not necessarily coincide with the periods of maximum amplitude of the accelerogram.

The method of solving dynamic problems for the "soil base – foundation – overhead construction" system presented and implemented in ASSR "VESNA" allows more precise modeling and therefore more efficient engineering designs for buildings by taking into consideration the specifics of dynamic interactions within such structures.

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