"Environmental Engineering" 10th International Conference
Vilnius Gediminas Technical University
Lithuania, 27–28 April 2017

Vibrations measurement of the funicular generated vibrations on Gediminas Hill North part slope

Šarūnas Skuodis[1], Kęstutis Kelevišius[2], Gintaras Žaržojus[3]

[1,3]Department of Geotechnical Engineering, Faculty of Civil Engineering, Vilnius Gediminas Technical University, Vilnius, Lithuania
[2]JSC "Vilniaus kogeneracinė jėgainė", Vilnius, Lithuania
[3]Department of Hydrogeology and Engineering Geology, Faculty of Chemistry and Geoscience, Vilnius University
E-mails: [1]sarunas.skuodis@vgtu.lt (corresponding author); [2]kestutis.kelevisius@le.lt; gintaras.zarzojus@vgtu.lt

Abstract. An experimental measurements of the funicular generated vibrations provided after Gediminas Hill North part slope landslide, which occurred on 2016. The geology of Gediminas Hill made up strata of Quaternary system late Pleistocene glacial and glaciofliuvial coarse and fine deposit.The purpose of this measurement was to determine, whether funicular generated vibrations during exploitation is the significant slope destabilizing factor. For vibration measurements in X, Y and Z directions were implemented equipment, developed by the authors. Measurements in 25 different points on the Gediminas Hill slope were perfirmed during funicular movement up and down. Analysis of obtained results revealed, that the highest vibration level mainly localized at the top of funicular foundations, wave energy is not large, propagating waves greatly dampens in the soil and there is no effect for general slope stability and the funicular exploitation was not the main reason of the occurred landslide in 2016 Spring.

Keywords: vibrations monitoring, slope landslide, vibrations measurement, ground vibrations.

Conference topic: Sustainable urban development.

Introduction

A landslide is a form of slope failure that includes a wide range of ground movements. Original slope stability could be affected by numerous contributing factors, which leads landslide to occur. Usually, pre-conditional factors build up specific sub-surface conditions that make the affected slope area to failure. One of the pre-conditional factors can be soil vibrations, which usually occurs due to human activity. Soil vibrations is a technical term that is being used to describe mostly man-made vibrations of the soil, in contrast to natural vibrations of the Earth studied by seismology. As an example, soil vibrations can be caused by explosions, construction works (Liyanapathirana, Ekanayake 2016), railway (Zhai *et al.* 2015; Connoly *et al.* 2015) and road transport, etc (Oettle *et al.* 2015).

In seismology, soil vibrations are associated with different types of elastic waves propagating through the soil. These are surface waves, mostly Rayleigh waves, and bulk longitudinal waves and transverse waves (Bessason, Erlingsson 2011) propagating into the soil depth. Typical frequency range for environmental soil vibrations is 1 – 200 Hz. Waves below 1 Hz are usually called micro-seismic, and they are normally associated with natural phenomena (e.g. water waves in the oceans). Environmental soil vibrations generated by rail and road traffic may cause annoyance to residents of nearby buildings both directly and via generated structure-borne interior noise. Very strong ground vibrations (generated by heavy lorries on bumped roads) may even cause structural damage to very close buildings or soil structures (e.g. retaining walls, slopes, etc.). Magnitudes of soil vibrations usually described in terms of particle vibration velocity (mm/s or m/s). Typical values of soil vibration particle velocity associated with vehicles passing over traffic calming road humps are in the range of 0.1 – 2.0 mm/s. Magnitudes of soil vibrations that are considered to be able to cause structural damage to buildings are above 10–20 mm/s. Knowledge and experience in understanding the causes of vibration effects of construction and industrial sources can be helpful in prevention of harmful ground and structure vibrations (Svinkin M. R. 2008).

Vibrations on Gediminas Hill North part slope is associated with funicular, which is taking the people to the top of Gediminas Hill. There is a possibility to get on top of the Hill and by foot. On the top of Gediminas Hill is still standing Gediminas tower, which is the remaining part of the Upper castle (Semaškaitė 2005) in Vilnius, Lithuania (the first brick castle part was completed in 1409). Funicular on Gediminas Hill was opened on 21 08 2003. This funicular is a short 71 m railway with wire rod. The rails are connected to 5 pile caps which is monolithically joining 4 pile foundations with diameter of 300 mm.

In this paper, authors want to present vibrations monitoring results in different distances from the funicular and to show the real affected area from funicular vibrations. According to obtained results it would be possible to approve or to deny funicular as a source of vibrations effect to the formation of occurred landslide in 15 02 2016 on Gediminas Hill North part slope.

Engineering geological conditions

The Gediminas Hill and Higher Castle along with Neris and Vilnelė rivers valleys and its slopes spreading from the confluence of rivers with other hills (Morkūnaitė *et al.* 2015), it's a part of territory of State Cultural Reserve of the Vilnius Castles. The reserve was established in 1997.

The most important stage of formation of mentioned hills (Baubinienė *et al.* 2015) began with the retreat of glaciers out of Lithuanian territory 18–22 thousands years ago. As a result the proglacial lakes starts disappearing, Neris and Vilnelė rivers due to erosion (Česnulevičius *et al.* 2006), abrasion and corrosion deepening the valleys and forming the terraces. During the formation of the Gediminas Hill, due to rivers work, all the time took place the slopes erosion that shaped steep hillsides. The hillsides erosion occurred from the North part by Neris and from the South and West parts by Vilnelė rivers. The east side of Hill was formed by groundwater work that erode a gulley. Here later, already in historical time, people have been artificially directed to Vilnelė riverbed.

The Gediminas Hill base have an East-West direction elongated elliptical shape. The long axis of elliptical shape is about 220 m, and the short axis is about 180 m. The top of the Hill is also elliptical shape, where the average length is 115 m, and the average width 55 m. The topography of top of the Hill is uneven. The South part of tip 10 m higher then North part. The slope of that Hill is steep, the angle is up to 40°. At present, the height of the Hill is nearly 48 m, measured from the Cathedral Square (Gaigalas, 1983; Gadeikis *et al.* 2016).

The geological information about Gediminas Hill has been obtained from borings and pits, and using a principle of analogy during which is comparing the data of others investigated hills in park. During the engineering geological investigations of Hill since 1955 up to now the total 431 borehole were bored. Only three deep boreholes were bored on a hilltop which ruched 35–45 m of depth, the others were up to 4–10 m of depth (Drumsta *et al.* 2010). The main geological structure of erosional Gediminas Hill consist of stratified different ice ages glacial deposits such as morainic and fliuvioglacial deposits, proglacial lake sediments. The basis of the Hill and bottom of Neris and Vilnelė rivers consist of the grey, very stiff morainic deposits (such as till and other mixtures of fine and coarse soils). These deposits occurred during the middle Pleistocene ice age – Dainava ice age (gIIdn), about 400 thousands years ago. The bottom of the Hill made up from very dense greenish-grey proglacial lake sediments (such as fine and silty sand) (lgIIdn). The thickness of the stratum 10–20 m. The middle part of Gediminas Hill is made up from brown stiff morainic deposits (such as till stratified with sand interlayers and lenses). These deposits was formed during the middle Pleistocene ice age – Žemaitija (Samogitia) ice age (gIIžm), about 250–175 thousands years ago. The thickness of the stratum is 10–15 m. The upper part of the Hill consist of fliuvioglacial deposits, which was deposited during the middle Pleistocene ice age – Medininkai ice age (fIImd), about 175–130 thousands years ago. The thickness of the stratum is 6–8 m. The deposits of this stratum consist of dense fine sand with silty sand interlayers. The superface of fliuvioglacial stratum is covered with terminal morainic deposits of middle Pleistocene Medininkai ice age (gtIImd). The deposits consist of thin layers (from 0.5–1.0 to 3.0–5.0 m of thickness) of firm and stiff brown till, with interlayers and lenses of sand. The most upper part of the Hill made up of the Holocene age artificial deposits (tIV). The thickness of the layer is 4–5 m. The slopes of the Gediminas Hill is covered by deluvium and artificial deposits. The thickness of layer is very uneven, on the hillside from 0.0 to 1.0 m or little more, at bottom of the slope up to several meters (Fig. 1) (Gaigalas, 1983; Guobytė 2008).

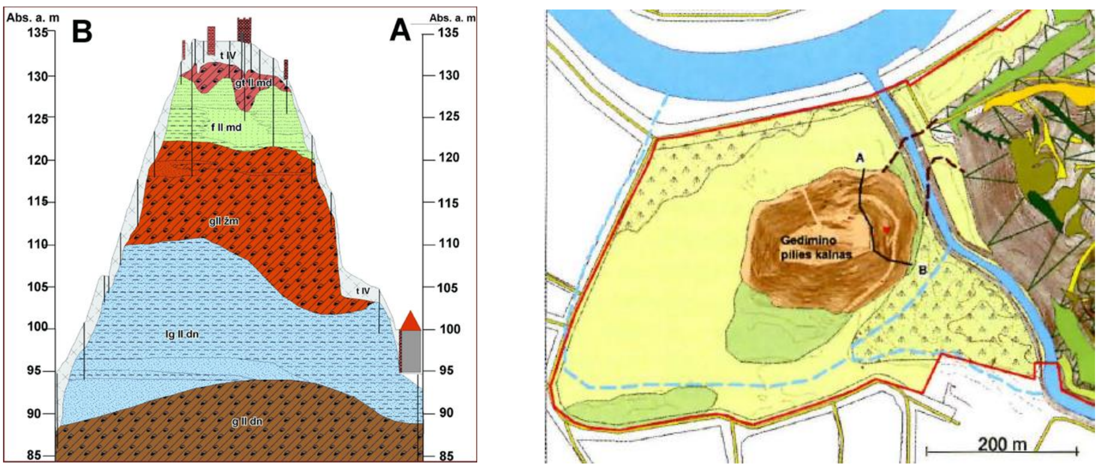

Fig. 1. Geological cross–section of Gediminas Hill (on left hand) and position and direction of cross–section line (on right hand) (Guobytė 2008)

The hydrogeological conditions of Gediminas Hill mostly depends on the amount of precipitation (Coppola *et al.* 2006; Dibal *et al.* 2014; Osman, Barahbah 2006). The part of precipitation water seep through the artificial (tIV), morainic deposits (gtIImd) and fliuvioglacial sand (fIImd) to the superface of Žemaitija (Samogitia) morainic deposits. Because the surface of this moraine is uneven and have the inclined hollow in centre, the groundwater discharged on the Hill east slope and forming the spring. Due to this phenomena in 2004 and 2008 (Čyžienė *et al.* 2012) there were two landslides in the Eastern part of Hill. Furthermore, the surface of stratum of Žemaitija morainic deposits has inclination to north direction (Fig. 1). Due to, the groundwater discharged on the North side of Hill, where landslide occurred in 2016.

Vibrations monitoring

Funicular vibrations area determination was started after 15 02 2016 Gediminas Hill North part landslide. The occurred landslide was only in 10 m distance from funicular. Vibrations monitoring was realised with 25 vibrations measurement points (Fig. 2).

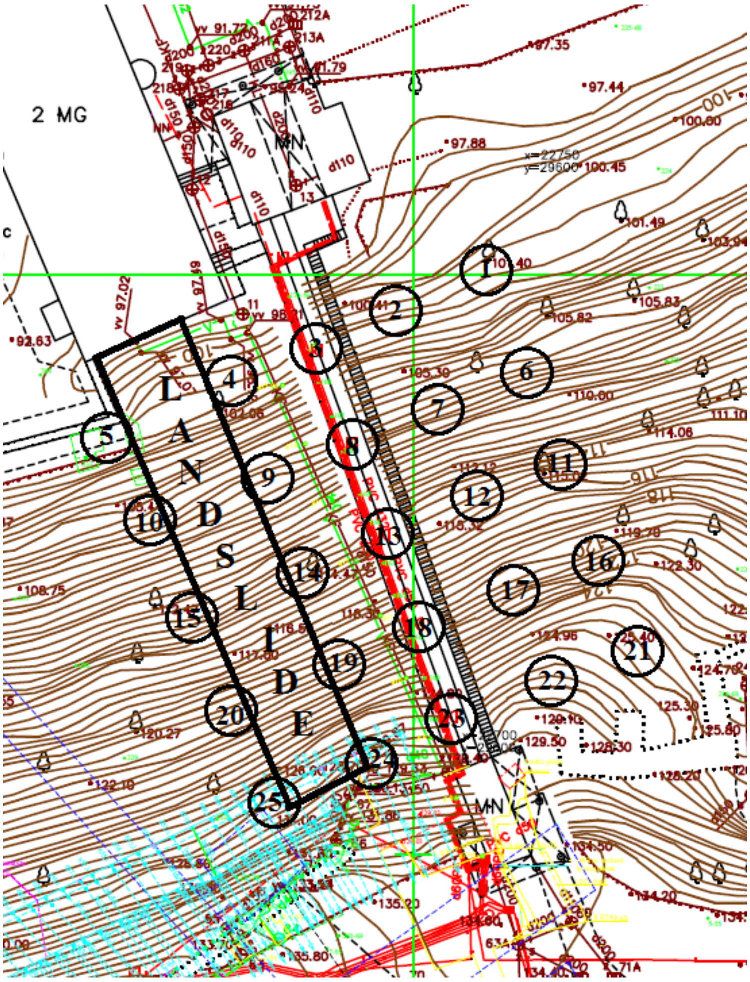

Fig. 2. Vibrations monitoring points and indication of landslide area

Distance between vibrations monitoring system indicators in horizontal plane was 10 m. At the vibrations monitoring area Gediminas Hill slope inclination varies from 35–40 to 60 degrees. Distance between horizontal vibrations monitoring system indicators rows was 11.80 m corresponding to funiculars pile foundations position (3; 8; 13; 18; 23 point in Fig. 2). In total, vibrations monitoring covered 1888 m^2 area, continuously recording soil surface vibrations when funicular was moving up and down. Surface vibrations in an elastic half-space are related to different waves types and traveling directions (see Fig. 3–4).

Vibrations monitoring was carried out measuring accelerations at the soil surface in 3 directions: X, Y and Z. In total 368 results were logged every second in all directions. Measured particles accelerations cross sensitivity does not exceed ±3 %. Acceleration sensor was connected to vibrations monitoring system corpus rigidly and jacked into the soil surface ~20 cm. Rigid connections between sensor-corpus-soil does not created low frequency damping component. All data from acceleration sensor transferred to data logger and only then sent to computer.

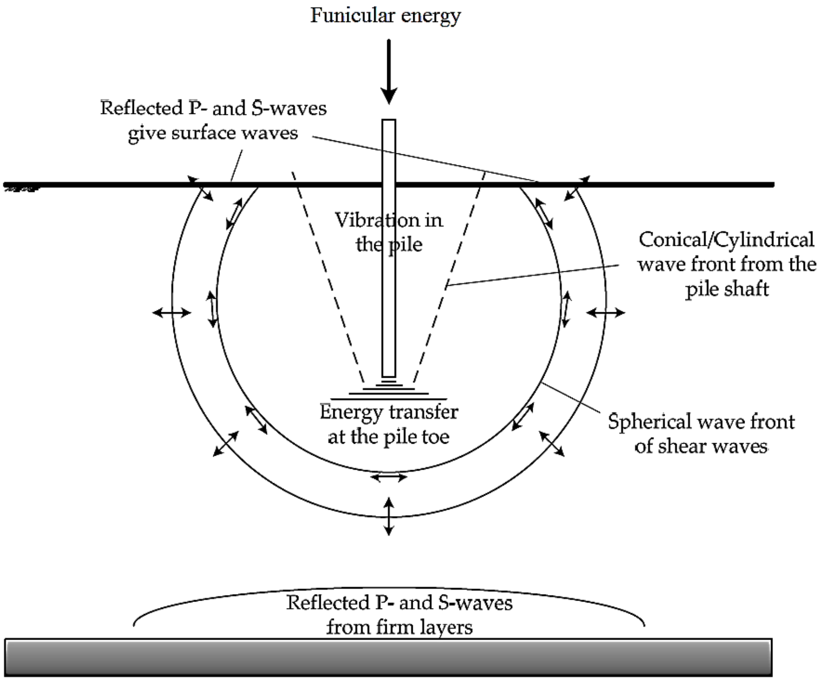

Fig. 3. Improved Deckner (2013) schematic representation of different wave types that can be generated from funicular

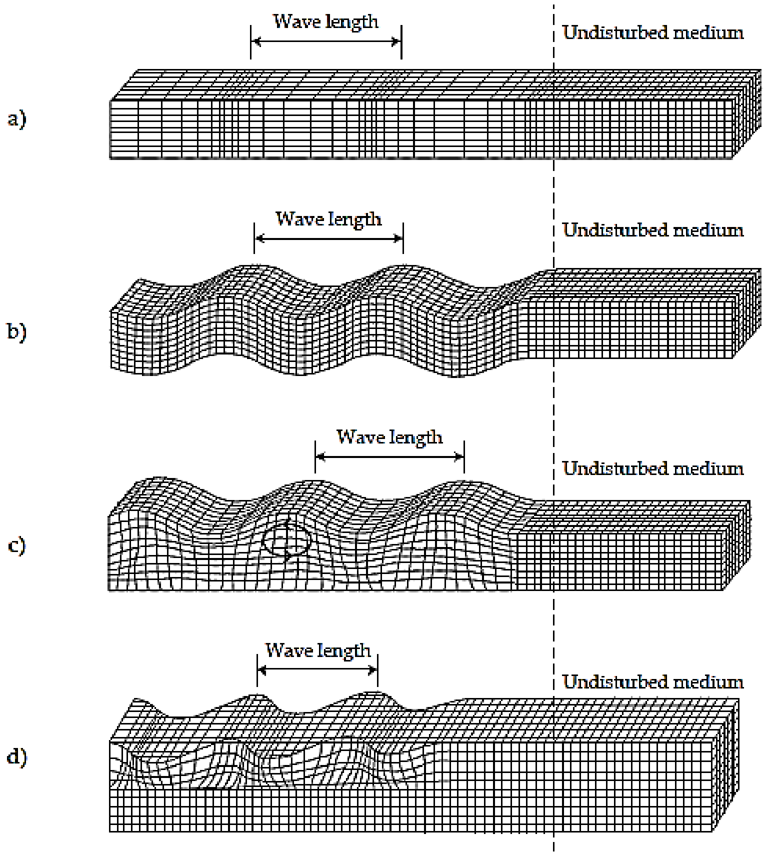

Fig. 4. Displacement characteristics of different wave types: a) P-wave, b) S-wave, c) R-wave and d) L-wave (see Deckner 2013)

Analysis of obtained results

Worst case scenario of all 25 vibrations measurement points (see Fig. 2) was chosen for determination of critical acceleration. Obtained maximum accelerations amplitude (see Fig. 5) was in vibrations measurement poit No. 23

(see Fig. 2), where accelerations in X axis min/max -0.985/1.014; in Y axis min/max -1.686/1.995 and in Z axis min/max -0.679/0.691.

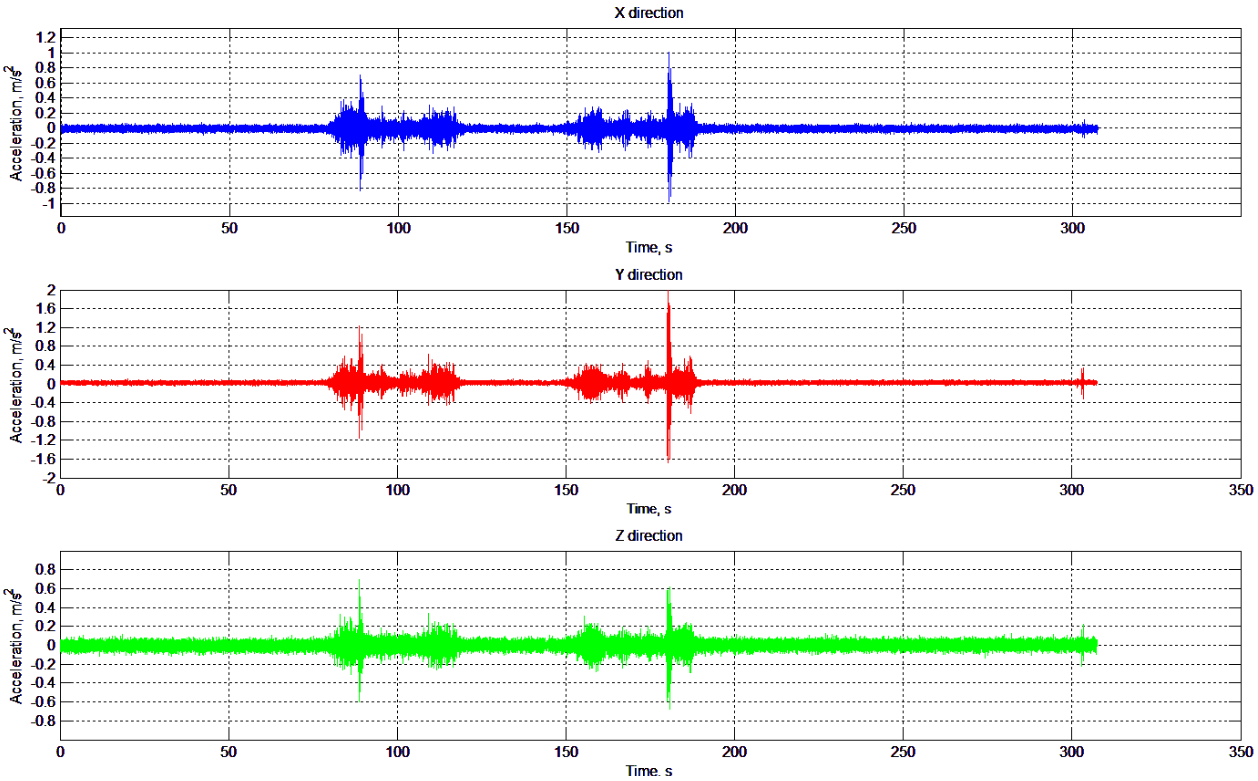

Fig. 5. Obtained maximum accelerations in vibrations monitoring area

Care and attention must be given to below guidelines because importing graphics pa ckages can often be problematic. Analysis of obtained results planned to be performed using static and dynamic slope stability. Static slope stability has to be performed in order to determine overall slpe stability and to determine critical acceleration for dynamic slope stability. Static slope stability analysis was performed using infinite slope method. That kind of method was chosen because type of Gediminas Hill occured landslinde mechanism (Fig. 6).

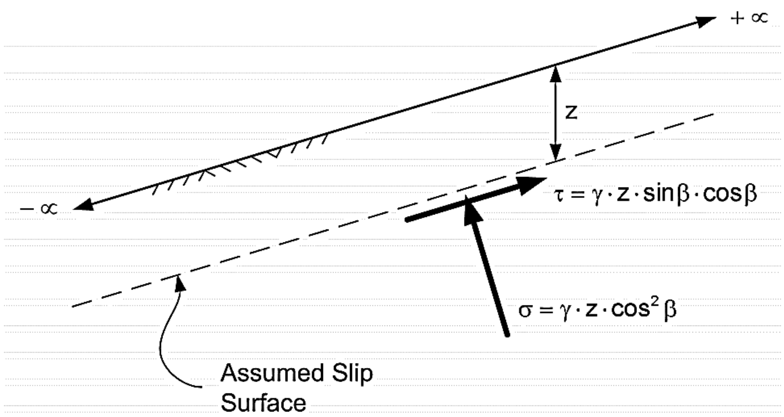

Fig. 6. Infinite slope failure (CECW-EW 2003)

Surface soil type of landslide and below it, can't be classified. It is mix of cohesionless and cohesive soil i.e. in one top layer can be found pure sand, in another – clay (see Fig. 1). Looking at overall picture of landslide soil, it is not homogenous cohesive soil. Considering this assumption the factor of safety (*FS*) for an infinite slope with seepage can be expressed as follows (Bolton 1979):

$$FS = \frac{(\gamma' - \gamma_w) \cdot \tan \alpha_s \cdot \tan \beta \cdot \tan \varphi'}{\gamma_{sat} \cdot \tan \beta}, \qquad (2)$$

where: γ' – submerged ($\gamma' = \gamma_{sat} - \gamma_w$) unit weight of soil (kN/m^3); γ_{sat} – saturated unit weight of soil (kN/m^3); γ_w – unit weight of water (kN/m^3); α_s – angle between the flow lines and the embankment face (degrees) (see Fig. 7); β – inclination of the slope measured from the horizontal plane (degrees); φ' – angle of internal friction expressed in terms of effective stresses (degrees).

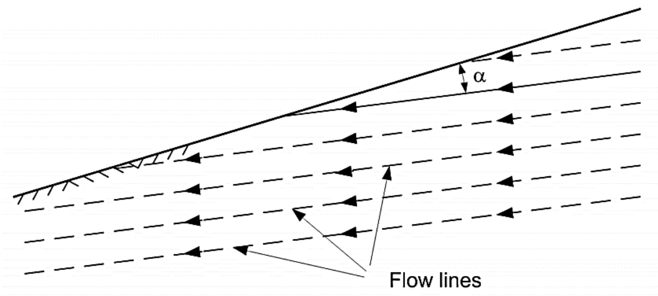

Fig. 7. Infinite slope with parallel water flow lines (CECW-EW 2003)

The cohesion (c') is assumed to be zero, because the infinite slope analysis is primarily applicable to cohesionless soils. According to Samtani and Nowatzki (2006), statement abuout global factors of safety is:
"A minimum factor of safety as low as 1.25 is used for highway embankment side slopes. This value of the safety factor should be increased to a minimum of 1.30 to 1.50 for slopes whose failure would cause significant damage such as end slopes beneath bridge abutments, major retaining structures and major roadways such as regional routes, interstates, etc. The selection of the design safety factor for a particular project depends on:
The method of stability analysis used ("For cut slopes in fine-grained soils, which can lose shear strength with time, a design safety factor of 1.50 is desirable"):

- The method used to determine the shear strength.
- The degree of confidence in the reliability of subsurface data.
- The consequences of a failure.
- How critical the application is." (Gediminas Hill case is loosing shear strength with time i.e. desirable safety factor is 1.50).

During calculation of *FS* two cases were assumed – 1) water flow lines after prolonged rain are parallel to Gediminas Hill slope surface and 2) no water flow inside slope soil. Slope angles from 45° to 60° for both FS calculation cases were used. Soil properties varies a lot and exact determination of landslide porperties is not possible, but looking at general picture (see Fig. 1), average soil properties are assumed. For this case internal firction angle of landslide is φ' = 28°. Calculated values are following: Case 1 when 45° slope inclination, obtained $FS_{45}^1 = 0.24$; Case 1 when 60° slope inclination, obtained $FS_{60}^1 = 0.14$; Case 2 when 45° slope inclination, obtained $FS_{45}^2 = 0.53$; Case 2 when 60° slope inclination, obtained $FS_{60}^2 = 0.31$.

Analysis results shows that calculated FS of Gediminas Hill is obviously smaller than required by Samtani and Nowatzki (2006). The slope stability greatly depends on water content. During sepage cohesive soils accumulates water content. As a consiquience of saturated cohesive soil sliding mass loses cohesiveness – binding areas and those cohesive places acts as sliding, but not binding areas. Permanent drainage or appropriate roots of vegetables are recomended for slope stabilizing to obtain higher cohesivenness (Malhotra, Lee 2008; Tosi 2007).

Dynamic slope stability method is based on Newmark's method. A key assumption of Newmark's method is that it treats a landslide as a rigid-plastic body: the mass does not deform internally, experiences no permanent displacement at accelerations below the critical level, and deforms plastically at constant stress along a discrete basal shear surface when the critical acceleration is exceeded. Other limiting assumptions commonly are imposed for simplicity (Jibson 2011):

- The static and dynamic shearing resistance of the soil are taken to be the same.
- The critical acceleration is not strain dependent and thus remains constant throughout the analysis.
- The upslope resistance to sliding is taken to be infinitely large such that upslope displacement is prohibited.
- The effects of dynamic pore pressure are neglected. This assumption generally is valid for compacted or overconsolidated clays and very dense or dry sands.

Newmark (1965) introduced a method to assess the performance of slopes during earthquakes that bridges the gap between overly simplistic pseudostatic analysis and overly complex stress-deformation analysis. Newmark's method models a landslide as a rigid block that slides on an inclined plane; the block has a known yield or critical acceleration, the acceleration required to overcome basal resistance and initiate sliding. An earthquake strong–motion record of interest is selected (Fig. 8), and those parts of the record that exceed the critical acceleration are integrated to obtain the velocity–time history of the block; the velocity–time history is then integrated to obtain the cumulative displacement of the landslide block (Jibson 2011).

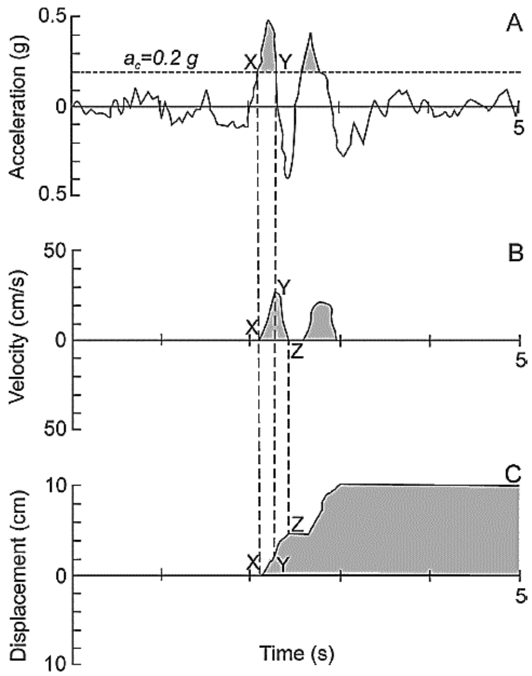

Fig. 8. Illustration of the Newmark integrations algorithm: A – earthquake acceleration-time history with critical acceleration (dashed line) of 0.2 g superimposed; B – velocity of the landslide versus time; C – displacement of landslide versus time; X, Y, and Z points are for reference between plots (Jibson 2011).

The critical acceleration a_c is presented in formula above:

$$a_c = (FS - 1) \cdot g \cdot \sin\alpha\,, \qquad (3)$$

where: a_c – *critical acceleration, g* – acceleration of gravity (m/s^2); FS – static factor of safety; α – angle from the horizontal of the sliding surface (Newmark, 1965).

Eq. (3) assumes that the seismic force is applied parallel to the slope. Unfortunately critical acceleration a_c can not be calculated because FS is smaller than 1. There is no need to calculate dynamic resistance of slope because it is not stable in static conditions. Surprisingly the only one part of the slope slided down. Conclusion can be made that there is a lot of cohesive soil that is in contac with natural soil of Gediminas Hill. The piles of funiculair are slope stabilizing elements also. More precise FS could be determined during general slope stability analysis taking into accoundsmall area homogenous soil FS.

Conclusions

The slopes of the Gediminas Hill is covered by deluvium and artificial deposits. The thickness of layer is very uneven, on the hillside from 0.0 to 1.0 m or little more, at the bottom of the slope up to several meters. Groundwater discharging on the North side of Hill, where landslide occurred in 2016. Determined maximum accelerations was in vibrations measurement point No. 23. Calculated static factor of safety for an infinite slope with seepage is less than 1. In that case there is no possibilities to calculate dynamic slope stability. Only one part of the slope slided down where a lot of technogenic soil / mould is filled. The rest of slope parts are still stable, because funiculairs piles are reinforcing slope and there is still left some cutted trees roots in the top soil layers. Trees roots are stabilizing slope and increases soil cohesiveness by providing drainage and anchorage. Summarizing all analysed factors, funiculair vibrations are not the main factor of Gediminas Hill landslide in 2016 Spring, because slope stability analysis shows slope collapse even under static analysis. The highest probability of occured landslide is due to lost local cohesion of the technogenic soil. More over, landslide occured at the nighttime when funiculair was not in operation.

Acknowledgements

An equipment and infrastructure of Civil Engineering Research Centre of Vilnius Gediminas Technical University was employed for current investigations.

Disclosure statement

Authors declare that they have any competing financial, professional, or personal interests from other parties related with manuscript.

References

Baubinienė, A.; Morkūnaitė, R.; Bauža, D.; Vaitkevičius G.; Petrošius, R. 2015. Aspects and methods in reconstructing the medieval terrain and deposits in Vilnius, *Quarternary International* 386: 83–88.

Bessason. B.; Erlingsson, S. 2011. Shear wave velocity in surface sediments, *Jökull* 61: 51–64.

Bolton, M. 1979. A Guide to Soil Mechanics. A Halstead Press Book, John Wiley and Sons, New York.

CECW-EW 2003. *Engineering and design slope stability, manual No. 1110-2-1902,* U.S. Army Corps of Engineers Washington, DC, USA.

Connoly, D. P.; Costa, P. A.; Kouroussis, G.; Galvin, P. Woodward, P. K.; Laghrouche, O. 2015. Large scale international testing of railway ground vibrations across Europe, *Soil dynamics and Earthquake Engineering* 72(2015): 1–12.

Coppola, L.; Nardone, R.; Rescio, P.; Bromhead, E. 2006. Reconstruction of the conditions that initiate landslide movement in weathered silty clay terrain: effects on the historic and architectural heritage of Pietrapertosa, Basilicata, Italy, *Landslides* 3:349–359.

Česnulevičius, A.; Bautrėnas, A.; Morkūnaitė, R.; Čeponis, T. 2006. Intensity of erosion processes in urbanization territories of Vilnius, *Geomorphology* 42: 1–6.

Čyžienė, J.; Minkevičius, V.; Mikulėnas, V.; Satkūnas, J. 2012. Geohazard description for Vilnius. PanGeo D7.1.28.

Deckner, F. 2013. *Ground vibrations due to pile and sheet pile driving – influencing factors, predictions and measurments*: Licentiate thesis. Royal Institute of Technology.

Dibal, J. M.; Ramalan, A. A.; Mudiare, O. J.; Igbadun, H. E. 2014. Scenario studies on effects of soil infiltration rates, land slope, and furrow irrigation characteristics on furrow irrigation–inducted erosion, *International Scholary Research Notices* 2014:1–6.

Drumsta V., Trumpis G., Grigonienė L., 2010. Gedimino kalno ir Aukštutinės pilies Vilniuje, (unikalus kodas 141) atliktų geologinių ir hidrogeologinių tyrimų archyvinės medžiagos sąranka, analizė ir išvados, *V. Į. Lietuvos paminklai.* 34 p.

Gadeikis, S.; Dundulis, K.; Daukšytė, A.; Gadeikyė, S. 2016. The Cathedral of Vilnius: Problems and features of natural conditions, in *13th Baltic Sea Geotechnical Conference*, 22–24 September 2016, Vilnius, Lithuania.

Gaigalas, A. 1983. Vilniaus pilies kalno sluoksniai, *Mūsų gamta*, 8: 6–7.

Guobytė R. 2008. Vilniaus pilių teritorijos egzotiškasis reljefas ir gelmių sandara, *Lietuvos pilys* 3: 34–35.

Jibson, R. W. 2011. Methods for assessing the stability of slopes during earthquakes–A retrospective, *Engineering Geology* 122: 43–50.

Liyanapathirana, D. S.; Ekanayake, S. D. 2016. Application of EPS geofoam in attenuating ground vibrations during vibratory pile driving, *Geotextiles and Geomembranes* 44(2016): 59–56.

Malhotra, S.; Lee, T. S. 2008. Reinforcement of slopes for seismic stability, in *6th International Conference on Case Histories in Geotechnical Engineering*, 11–16 August 2008, Arlington, VA, USA.

Melo, C.; Sharma, S. 2004. Seismic coefficients for pseudostatic slope analysis, in *13th World Conference on Earthquake Engineering*, 1–6 August 2004, Vancouver, B. C., Canada.

Morkūnaitė, R.; Baubinienė, A.; Vaitkevičius, G. 2015. Geographical–historical interpretation of natural conditions in the reconstruction of Vilnius city formation and development, *Geologijos akiračiai* 3: 15–22.

Newmark, N. M. 1965. Effects of earthquakes on dams and embankments, *Geotechnique* 15: 139–159.

Oettle, N. K.; Bray, J. D.; Dreger, D. S. 2015. Dynamic effects of surface fault rupture interaction with structures, *Soil dynamics and Earthquake Engineering* 72(2015): 37–47.

Osman, N.; Barahbah, S. S. 2006. Parameters to predict slope stability–soil water and root profiles, *Ecological Engineering* 28(2006): 90–95.

Samtani, N. C.; Nowatzki, E. A. 2006. FHWA-NHI–06-088, Soils and foundations reference manual, Volume I.

Semaškaitė, I. 2005. Lietuvos pilys ir dvarai: istorijos ir legendos: architektūros bruožai. Algimantas, Vilnius.

Svinkin, M. R. 2008. Soil and structure vibrations from construction and industrial sources, in *6th International Conference on Case Histories in Geotechnical Engineering*, 11–16 August 2008, Arlington, VA, USA.

Tosi, M. 2007. Root tensile strength relationships and their slope stability implications of three shrub species in the Northern Apennines (Italy), *Geomorphology* 87(2007): 268–283.

Yin, Y. 2014. Vertical acceleration effect on landslides triggered by the Wenchuan earthquake, China, *Environmental Earth Sciences* 71(11):4703–4717.

Zhai, W.; Wei, K.; Song, X.; Shao, M. 2015. Experimental investigation into ground vibrations induced by very high speed trains on a non–ballasted track, *Soil dynamics and Earthquake Engineering* 72(2015): 24–36.