Influence of vibrations on structures

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One type of occasional structural load is a seismic load. Earthquakes and blasts are typical sources of vibrations, but vibration generated during urban tunnel construction can represent a significant problem. Evaluation of the harmful impact of vibrations transmitted through rock massif into buildings is solved using experimental measurements, detailed analyses of measured signals, knowledge of geological pattern and constructional analysis.

Seismic load of structures due to earthquakes is solved using the EUROCODE 8 standard. The earthquake movements at a certain location on the surface are determined by an elastic response spectrum to the ground acceleration. Eurocode 8 puts emphasis especially on the robust foundations and simplicity of construction systems. It is also mentioned vibration effect on historical buildings and effect under the surface, for example, in mine spaces. Historical structures are usually even more prone to vibration damage than, for example, typical wood-frame homes. The greater concerns over historic structures arise from the design, structure age, building materials and building methods used. The peak values of vibration generated by earthquake decrease with depth; the decrease is faster in shallow layers compared with the deeper part. Technical vibrations differ from natural earthquakes, for a comparable value of maximum vibration amplitudes, especially in the frequency range of the signal and mostly its duration. Evaluation of technical seismicity is more complicated because there are usually used national standards.

To document some common information about vibration effects on structures, some experimental measurements are presented. Examples of real wave patterns document common shapes and also signals with significant resonant vibrations. Very interesting is an example of resonant vibration that was generated as the influence of basin structures on the shape of wave patterns due to quarry blasts. To obtain complete information, measurement system has to keep sufficient parameters, especially the frequency range of the whole seismic channel, sampling frequency, and proper anchoring of the sensor. The basic methodology for evaluation of vibration on structures is outlined.

Keywords: *seismic load, seismic standard, Eurocode 8, earthquake, technical seismicity*

Introduction

The basic objective of all activities in the designing and realisation of structures must be to create a quality environment suitable for the intended purpose of the structure, while this quality should be maintained over the entire expected life of the structure. The basic requirements for structure construction are (according to Merritt and Ricketts, 2001; Macdonald, 2001; Chudley and Greeno, 2014):

- Architectural requirements;
- Structural static requirements;
- Resistance to external influences;
- Welfare and hygiene requirements for the indoor environment;
- Operational safety requirements;
- Technology requirements;
- Economic requirements;
- Environmental requirements.

A load of structures can be classified as follows:

- Occasional loads (long-term, short-term, extraordinary)
  - Payloads;
  - Climate loads;
  - Snow load;
  - Wind load;
  - Frost load;
  - Load from forced strains;
  - Temperature load;
  - Load by rheological material changes;
  - Load by deteriorating support;
  - Mounting load;
  - Seismic load;
  - Pressure waves;

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- o Emergency load.

- Permanent loads
  - o Load by the weight of the structure;
  - o Pressure load;
  - o Preloading.

In order to assess the seismic load (vibrations) of structures, we need to determine the safe boundary that does not break the object or release the rock. The occurrence of new cracks or the widening of existing cracks, in the case of structures such as a drop of mortar or plaster or falling-off rock fragments, is considered a failure in both cases mentioned above (for example, Tripathy et al., 2016; Zeigler, 2018). It is necessary to prevent catastrophic failure, i.e., collapsing the structure (Fig. 1) or rocking off, at all times (for example, Towhata, 2008; Villaverde, 2009). On the other hand, cosmetic (light) damage is allowed in some cases if it does not compromise the safety of the structure. To do this, it is necessary to know the appropriate criterion for assessing the vibration effects that can be measured and potentially extrapolated on the endangered structure. Further, the degree of violation must be distinguished more precisely, and finally, we should have the possibility of a preliminary estimate of the vibration effects. The assessment of seismic load of structures often results from the measurement of vibrations at the reference standing place (for example, standard ISO 4866). In order to assess the response of structures, records should be obtained for such intensity of vibrations that evoke measurable effects on the structures. Vibration records have to be realised in an adequate time and frequency ranges (for example, Scherbaum, 1994).

Fig. 1. A man walks past a collapsed building in Dabandikhan in Sulaimaniya Governorate, Iraq. (Photo by Ako Rasheed, Reuters; Received from http://www.interaksyon.com/toll-from-iran-iraq-quake-breaches-450/).

As an example, typical damages in a masonry building are presented in Fig. 2 (according to www.earth-auroville.com/index.php). Structural elements, such as walls, columns, and beams, are only bearing the weight of the building and the live load under normal conditions: mostly compression forces for the walls and columns, and vertical bending for the beams. Under dynamic load, they also have to withstand horizontal bending and shear forces, and extra vertical compression forces. Several types of cracks are possible to define (Fig. 2). It is necessary to point out that small amounts of cosmetic cracking can arise from slight settling, ground movement, temperature, and humidity cycling, and even, in extreme cases (hurricanes, tornadoes), wind loading (Zeigler, 2018).

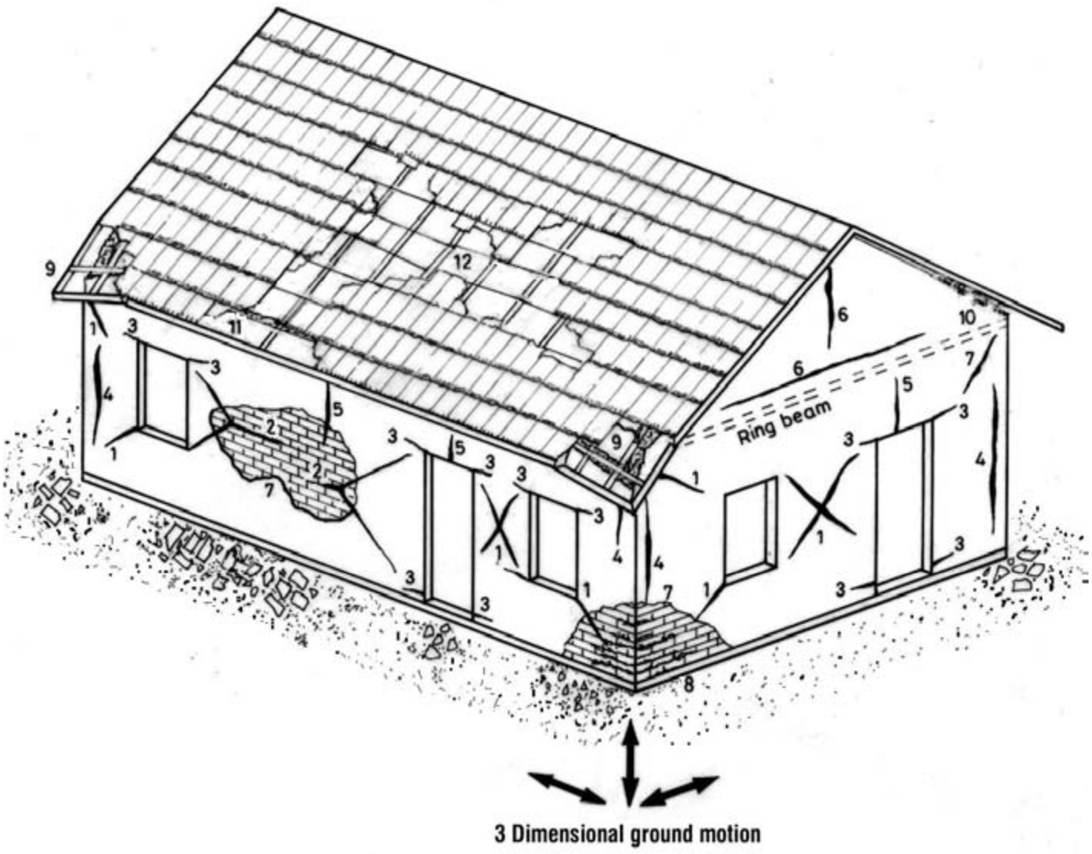

Fig. 2. Typical damages in a masonry building (according to www.earth-auroville.com/index.php)
1: Diagonal shear crack of piers, 2: Horizontal shear crack of long pier, 3: Bending cracks at feet and lintels, 4: Bending crack of wall (bad corner bond), 5: Bending crack of spandrel, 6: Bending crack of gable, 7: Plaster peeling off, 8: Crushing of weak masonry under vertical ground motion, 9: Damage of corner eaves under vertical ground motion, 10: Badly anchored roof, pulled out by vertical ground motion, 11: Falling of tiles from the roof eave, 12: Damage of tiles roof with shear (roof not braced).

To properly support a structure in response to whatever loads may be applied to it, a structure must possess four properties: it must be capable of achieving a state of equilibrium, it must be stable, it must have adequate strength, and it must have adequate rigidity (Macdonald, 2001). The achievement of stable equilibrium has been shown to be dependent largely on the geometric configuration of the structure and is, therefore, a consideration which affects the determination of its form. A stable form can almost always be made adequately strong and rigid, but the form chosen does affect the efficiency with which this can be accomplished. Therefore, collaboration has always been required between architects and those who have the technical expertise to realise building construction (for example, Merritt and Ricketts, 2001; Chudley and Greeno, 2014).

Every structure has vibration frequencies and mode shapes that are called "natural frequencies“ that can be found by using analytical methods. Calculation of these frequencies and their mode shapes are important to solve the vibration induced engineering problems. However, complex shaped objects can only be analysed by numerical methods. In particular, finite element methods (FEM) and boundary element method (BEM) enable to investigate the natural frequencies and mode shapes of complex structures by idealising them into computable small parts (for example, Berr, 2003; Hori, 2006; Chakraverty et al., 2012). Vibration analyses can be divided into two main parts. These are natural frequency and mode shape extraction and forced vibration analysis. By natural frequency analysis, the object's natural frequencies are obtained. A frequency of a periodic force which is applied to this object could be near to one of the object's natural frequencies. If so, that frequency is excited, and the structure starts to vibrate in its mode shape and natural frequency. If excitation frequency comes across the structure's natural frequency, "resonance" event occurs. In many case resonance is undesirable, and either excitation frequency or structure's natural frequency should be changed (http://www.mesh.com.tr/vibration-analyses.html).

The examples of analysis of the impact of vibrations on structures are presented in this paper, which is based mainly on the Eurocode 8 standard. Although this standard deals with the design of earthquake resistant structures, general rules apply to technical vibrations too. It is also necessary to point out the most common fundamental differences between natural and technical vibrations (at comparable amplitude values), which is especially the frequency range of the signal and mostly its duration. International standard ISO 4866 introduces frequency range 0.1 – 30 Hz and velocity amplitude range 0.2 – 400 $m.s^{-1}$ for earthquakes. For quarry blasts, it introduces frequency range 1 – 300 Hz and velocity amplitude range 0.2 – 500 $m.s^{-1}$, and for other technical sources frequency range usually 1 – 100 Hz (up 1 kHz for machines) and velocity amplitude range up 0.2 – 50 $m.s^{-1}$.

Several examples of vibration records realised in a different type of structures, as mentioned at the end of this paper, document variability of structure responses.

Vibration movement

Theory of vibration movement, especially harmonic vibration, is commonly known and it is described in many textbooks, including seismological literature (for example, Bullen and Bolt, 1985; Kulhánek, 1990; Doyle, 1995; Udías, 1999; Shearer, 2009). The ground motions that are produced by earthquakes can be completely described by six components of motion, i.e., three translational components and three rotational ones, and by deformation (for example, Båth, 1979; Teisseyre et al., 2006; Graizer, 2006). Usually, only translational components are used for interpretation. Rotational components have been known for several centuries. However, it is only during the last two decades that greater attention has been dedicated to precise measurements of these. Although rotational components usually have small values, several studies have shown the importance of these components in seismological analyses and engineering applications (for example, Lee et al., 2009; Knejzlík et al., 2012; Kaláb et al., 2013).

Seismic movement can be described by the time variations of the ground acceleration and its associated parameters (velocity, displacement). The maximum (peak) ground acceleration, duration, and frequency content of earthquake can be obtained from an accelerogram. The seismic movement must be composed of three simultaneously acting accelerograms (calculated, actual, and simulated) in the case when a spatial model is to be considered. If particular constructions are evaluated, it is possible to describe the ground movement as a function of location and time. The damage potential of a given vibration is often assumed, even by those who do vibration monitoring, to be governed only by the maximum ground velocity of the vibration. However, the detailed frequency component makeup of the vibration, its duration and the number of times it is repeated all contribute to its potential for causing damage (for example, Lyubushin, 2007; Lyubushin et al., 2012; Zeigler, 2018).

We distinguish primary and secondary members as regards vibration of structures. A certain number of supporting elements can be designed as secondary seismic members, which do not form part of a seismic load-bearing structure. These are all parts of structures that hold something up but are not crucial to the building's structural integrity. The strength and stiffness of these members against seismic actions shall be neglected. These members and their connections shall be designed and constructed to maintain support of gravity load when subjected to the displacement generated by the most unfavourable seismic design condition. All structural elements that are not designed as secondary are considered as primary seismic members (see Eurocode 8). It means everything without which the structure will not stand up (typically columns, braces, and beams in steel constructions, add shear walls and slabs in concrete constructions). It is considered to be part of the load-bearing system that is resistant to transverse forces (for example, Iervolino et al., 2008).

As mentioned above, vibration load can be expressed in several ways. The earthquake movement (i.e., natural origin) at a certain location on the surface is determined by an elastic response spectrum to the ground acceleration (for example, Gupta, 1992; Viskup et al., 2005). The shape of the response spectrum is assumed to be the same for both seismic loads levels (i.e., the ultimate limit state and damage limitation state). The horizontal load is described by two independent perpendicular components (but with identical spectra). One or more different response spectra are used for all three components (occasionally a vertical component is also used) of the seismic load depend on the source parameters and magnitude of the earthquake. Also, more than one spectrum should be considered if earthquakes threatening the evaluated locality can come from directionally different source areas (for example, Bolt, 1999; Udías, 1999). In the last mentioned situation, different values of design acceleration values a_g will have to be defined for each spectrum type and earthquake. Response spectra (Fig. 3) are used to provide the most descriptive representation of the influence of a given earthquake on a structure (for example, Ali et al., 2017).

The guiding principles governing this conceptual design are (Eurocode 8):

- Structural simplicity;
- Uniformity, symmetry, and redundancy;
- Bi-directional resistance and stiffness;
- Torsional resistance and stiffness;
- Diaphragmatic behaviour at storey level;
- Adequate foundation.

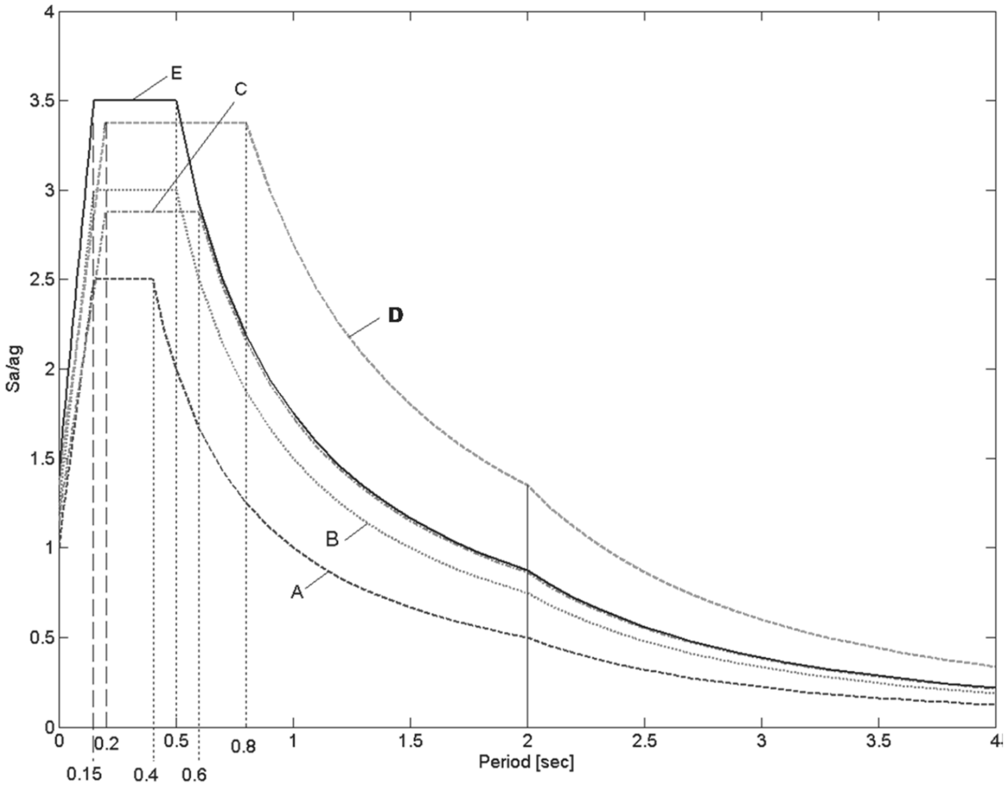

Fig. 3. Spectral shapes for main site classes (labelled as A – E). (Received from Iervolino et al., 2008).

In design calculations, structures are to be modelled, designed and modified according to the Eurocode rules. In terms of seismic design, the structures are categorised into regular and non-regular (which have an impact on the design model, the calculation method and the coefficient of ductility). The criteria for regularity in plan and elevation are defined.

Buildings are classified in 4 importance classes, depending on the consequences of a collapse for human lives, on the importance for public safety and civil protection in the immediate post-earthquake period, and the social and economic consequences of collapse (Eurocode 8). In the Czech Republic (National annex of the Eurocode 8), the following values of the importance factor γ are used: I = 0.8; II = 1.0; III = 1.2; IV = 1.4.

The Eurocode 8 puts emphasis especially on the robust foundation and simplicity of construction systems. This standard also allows differentiation of constructions according to their importance, their dimensions, and their mechanical action. Among other things, the standard specifies conditions for building site selection, soil parameters and also criteria that ground and foundation systems have to complete in seismic design situations. Eurocode 8 provides a simple quasi-static solution for ordinary buildings; seismic forces that already include the effect of motion are determined. Generally, horizontal excitation is used because vertical excitation is usually smaller, and structures are even more resistant to this direction because the design of structures respects their weight. Vertical loads can be of use both in the areas near the epicentres and in the case of long brackets or beams loaded with non-pillar columns.

It is also necessary to take into account that loads of buildings and structures by technical seismicity and their responses are evaluated according to, for example, Czech Technical Standard 73 0040 or Slovak Technical Standard STN EN 1998-1/NA/Z1. This evaluation is established using a class of resistance (A – F) and class of economic and social significance of buildings (U, I – III). Structures of A type are usually historical monuments and buildings, the oldest and poorly structures and also buildings with large plastic decoration; structures of B type are common masonry buildings, usually up to three levels and surface up to 200 m^2. Determination of resistance class depends also on the constructional technology and material used. From the constructional point of view, there are monolithic structures with resistance class E, framed structures with class D, half-timbered structures with class D, buildings up to three stories with class B and prefabricated panel structures with class C. Resistance class can be determined based on material used: stone - resistance class A, masonry - resistance class A, B, C, concrete - resistance class C, D, steel - resistance class D, E and steel concrete – E. Class of significance U represents structures with extraordinary economic and/or social significance (for example, dams, significant bridges …), following class I is represented by structures with great significance (for example, schools, churches…), classes II and III include structures with medium and small significances, respectively.

Historic structures (class A) are usually even more prone to vibration damage than typical wood-frame homes. The greater concerns over historic structures arise from the design, structure age, building materials and building methods used. Maintenance can be an issue in some cases, as well (Johnson and Hannen, 2015; Bongiovanni et al., 2017). For example, tower structures were often built by Romans to celebrate military victories. They played an important role in the reconstruction of some historical periods, but also in the study of the historical seismicity (for example, Bongiovanni et al., 2014). And further example, Clemente et al. (2002) analysed the experimental seismic behaviour of a bell tower damaged by the 1996 Reggio Emilia earthquake. Naturally, engineered, steel-reinforced buildings are more resistant to vibration damage than engineered, non-reinforced structures. All authors, not only describing historical structures loading, pointed out the importance of a multidisciplinarity approach to analyse and preserve given structure.

A most important article on amplitude versus depth relationship was published by Chinese researchers Hu and Xie in 2004. The following information was extracted from this article. The ratio of the amplitudes in the observed depth (underground) and the surface is used for the examination of changes in the value of vibration relative to depth (underground amplitude/surface amplitude). The amplitude ratio is calculated from the maximum values, which can be measured as the peak ground acceleration (PGA), the peak ground velocity (PGV) or the peak ground displacement (PGD). The reason for choosing the surface value as a comparative value is the following. In general, the value of the maximum surface amplitude is greater than in the underground, so if we use a larger value in the denominator, the relative error is reduced. Further, the surface records are much more frequent than from underground; therefore the values for the underground can be determined using statistical regression curves. The evaluation procedure is as follows: Firstly, we issue from the data recorded by a network of seismic stations; to study the effects of earthquakes depending on the depth, the recorded data are split into groups depending on the magnitude of the earthquake and peak amplitude. Secondly, it is possible to calculate the value of amplitude ratios for earthquakes of the same group. Based on this, the average size ratio of all the earthquakes of the same depth can be determined, thus obtaining the average amplitude ratios at different depths. Thirdly, the curves of average amplitudes for each group of earthquakes are determined using non-linear regression analysis. For research purposes, the value of the horizontal component amplitude is determined as the average of two components (NS and EW in the geographic positioning of seismometers).

The Dowding´s and Rozen´s study (1978 in Varnusfaderani et al., 2015) divides the damage to underground structures into three categories according to the effects of the earthquake: damage caused by vibrations, damage caused by faults and damage caused by disturbances due to the earthquake, for example, soil liquefaction or landslides. Varnusfaderani et al. (2015) also pay attention to the source mechanism of earthquakes. Manifestations of underground seismic activity can be divided into two categories:

- Vibration - changes in stress-strain conditions;
- Loss of stability - soil liquefaction, a fault in the rock massif, landslides.

Three types of deformation caused by seismic vibration appear in linear structures located in the underground (Owen and Scholl, 1981):

- Longitudinal axial deformation caused by pressure or tension;
- Bending of the direction axis of a tunnel;
- Deformation of the circular section into oval or frame deformation (racking).

Although the variation characteristics of PGA in different sites share some common features, as shown in Fig. 4, there are still some differences which can be summarised as follows. What is needed to say is that although the events are not quite enough in soil/rock site to account for the specific reduction characteristics of PGA, in order to illustrate the essential characteristics between different site conditions we still compared the two sites.

- The PGA decline velocity for soil/rock site is the most rapidly, for rock site it is the least rapidly, and for soil site, it is in the middle of the two sites.
- The variation of PGA with depth is affected by the magnitude of earthquakes and site geology. For soil site, the PGA decreases with the increasing of magnitude or intensity; for rock site, the declining extent of the larger earthquakes is more rapidly than that of the smaller's.
- Different from rock site, for soil site, there is a dramatic declination in the shallower layer.

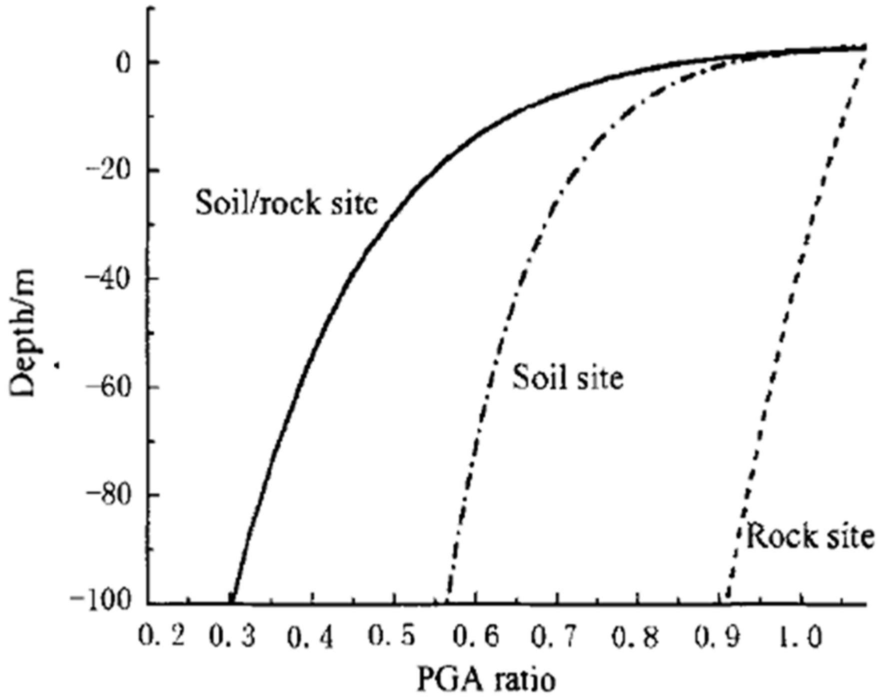

Fig. 4. The comparison of PGA ratios in different sites (from Hu and Xie, 2004).

Based on the results of Hu and Xie (2004), it drew some general conclusions:

- In general, the earthquake amplitude (PGA, PGV or PGD) decreases with depth, and the declining extent is more dramatic in shallower layers than that in deeper ones.
- The reduction of amplitude with depth is affected by the magnitude and site geology. In general, for soil site, the declining extent decreases with the increment of magnitude as well as the amplitude.
- For soil site, as shown in Fig. 5, the decline velocities of PGA, PGV and PGD decrease in sequence. For soil/rock site, the decline velocities of PGA, PGV, and PGD are similar to each other.

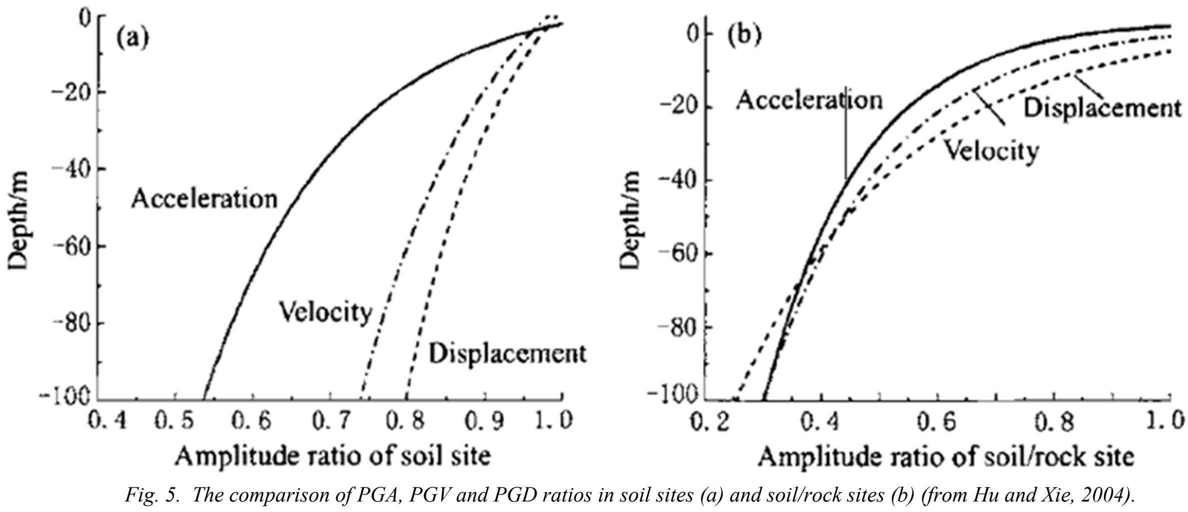

Fig. 5. The comparison of PGA, PGV and PGD ratios in soil sites (a) and soil/rock sites (b) (from Hu and Xie, 2004).

Results imply that PGA decreases with depth and the decline mainly focus on shallower layers. For example, the PGA in depth of 25 m decreases to 1/2 that of the surface. Moreover, as we all known in seismic response analysis, the input motion for structures are generally deduced from the design intensity of surface, and then the surface motion, like PGA, PGV, PGD or time histories, are put to the bottom of the buildings. Obviously, it is inappropriate for that the depth of burial of underground structure or high-rise buildings are always more than 10 m, and the general ideas of doing so are that it would lead to an overestimate of seismic response.

The issue of manifestations of earthquakes in underground mines is in the interest of responsible workers for many years. As an example, we present the findings which were published in a comprehensive study already in 1978 by Pratt, Hustrulid and Stephenson of Utah.

Stevens (1977 in Pratt et al., 1978) summarises the nature of earthquakes and lists numerous examples concerning the earthquakes manifestations on underground structures. He has several general conclusions:

- Effects on mines are less severe than surface effects.
    - Severe damage is inevitable when a mine or tunnel intersects a fault along which movement occurs during an earthquake;
    - Mines in the epicentre region of strong earthquakes but not crosscut by fault movement may suffer severe damage by shaking. Stevens did not define the word severe quantitatively;
    - Mines outside of epicentre regions are likely to suffer little or no damage from a strong earthquake;

- Damage to mines is most insignificant when they are located in highly competent, unweathered rock; greatest damage occurs in mines found in loose unconsolidated or incompetent rock. This is due to the diminished effect of shaking in competent rock; unconsolidated sediment is much more susceptible to damage caused by vibration.

Principal conclusions developed in this study are:

- There are very few data on damage in the subsurface due to earthquakes. This fact itself attests to the lessened effect of earthquakes in the subsurface because mines exist in areas where strong earthquakes have done extensive surface damage.
- More damage is reported in shallow tunnels near the surface than in deep mines.
- In mines and tunnels, large displacements occur primarily along pre-existing faults and fractures or at the surface entrance to these facilities.
- Data indicate vertical structures such as wells and shafts are less susceptible to damage than surface facilities.

Parametric studies were carried out to investigate the influence of the desired variables on the dynamic behaviour of an underground structure while earthquake loading (Serati and Moosavi, 2010). To perform a sensibility analysis of a particular parameter, a suitable model based on all important concepts of dynamic analysis was created. Then by changing the desired parameter in the selected range and keeping all other conditions of the model (such as model dimension, element size, etc.) constant, acceleration, displacement and velocity histories around the underground structure were calculated. Finally, the spectrum was obtained so that its independent variable was the desired parameter and its dependent variable was the maximum amplitude of velocity, displacement or acceleration history. Examining the obtained spectrum shows how changing in the desired parameter can affect dynamic records amplitude around the basic model of the underground structure. The greatest impact on the dynamic behaviour of the underground of structures during an earthquake in terms of geotechnical parameters is seen on the elastic modulus and density of the rock massif. The most important parameters, however, are the geometric parameters, i.e., the depth and diameter (the size) of underground structures.

According to Dowding and Rozen (1978), damage to shallow tunnels due to earthquakes occurs upon the excess of the certain value of acceleration (ground velocity) measured on the surface. The "no damage" limit lies below the level of 1.9 m.s^{-2} (approx. 0.2 m.s^{-1}); the level of lesser damage lies in the range of 1.9 m. s^{-2} (approx. 0.2 m.s^{-1}) to 5.0 m.s^{-2} (approx. 0.91 m.s^{-1}). The authors consider the values of the influence of vibrations in shallow underground areas as about half the value measured on the surface. Then the limit values for underground spaces are approximately 0,9 m.s^{-2} (0,1 m.s^{-1}) for no damage zones, and limit values for lesser damage in the range of 0,9 m.s^{-2} (0,1 m.s^{-1}) to 2,5 m.s^{-2} (0,45 m.s^{-1}). Singh (2002) documents different limit values for the occurrence of damages in a mine as a result of vibrations induced by blasting explosives on the surface. According to this study, the first damage can arise after reaching a ground velocity of 0.05 m.s^{-1} in rocks of very poor quality (RMR = 20 - 30). Different limit values refer to different underground structures and different sources of vibration, i.e., particularly to the frequency range of vibrations, local geological pattern, the shape and geometry of the underground spaces, reinforcement of underground structures and other parameters. Model experiments in various underground structures have shown that they are resistant to vibration. However, the existence of seismic discontinuity makes these structures exceptionally susceptible to collapse, especially in the case of shallow underground structures. Numerical modelling of the dynamic response of underground gas storage in caverns after the seismic load is stated in Wang et al. (2014). They use the modelling program FLAC3D, and they consider the size of the induced acceleration and the duration of vibrations as main factors.

New findings of the effect of technical vibrations

As mentioned in the introduction, technical vibrations differ from natural earthquakes, as regards the comparable value of maximum vibration amplitudes, especially in the frequency range of the signal and mostly its duration. The most usual attribute of technical vibrations is that they are often repeated shocks or periodical signals. In case of corresponding damage of construction, it is necessary to take into account the fact that repeated load, even if it does not reach critical values, can seriously debase the technical conditions of the structures, especially if they are already fissured and/or cracked. In addition, it is necessary to consider higher frequencies, whether the generated frequencies do not match the eigenfrequency of some of the primary or secondary seismic members of the structure. Then, resonant vibration is generated, and a significant increase in the probability of cracks occurring is expected.

The most intensive vibration effect is generated by a blast of explosives (Fig. 6). Blasts are represented by short but usually very intense impulses. As referred in ISO 4866 Standard, the frequency spectrum of the seismic records is continuous and includes frequencies ranging from lower values to very high values - usually 1 to 300 Hz. Wide frequency range of blasting depends on the properties of the disintegrated material, explosive properties, and blasting technology. The frequency spectrum of the seismic record of blasting is further significantly influenced by the environment, in which the waves pass through; higher frequency components are in the rock mass attenuated faster with increasing distances (for example, Barton, 2006; Banerjee and Kumar, 2016).

Fig. 6. Blast in Dewon quarry, Jarnoltowek, Poland, 2012. (Photo: author).

The evaluation of the effect of technical vibrations is usually based on measured values of ground velocity or acceleration; the maximum values can be calculated from empirical relations in some types of technical seismicity. To obtain experimental values, sensors are placed in the evaluated structures, either in cellars or on the lowest floor, and the enclosure load-bearing wall.

Usually, the load of structures generated by blasting vibrations is evaluated according to the maximum ground velocity amplitude (or acceleration) and the frequency of the prevailing vibrations (for example, CSN 730040 – Czech Republic, STN EN 1998-1/NA/Z1 – Slovak Republic, DIN 4150 – Germany, PN-B-02170:2016-12 – Poland …). An empirical relationship is formed which represents the dependence of the maximum ground velocity amplitude V_{max} on the total weight of the charge (or the weight of the charge fired at the one-time stage) Q and distance l (for example, Dojčár et al., 1996; Tripathy et al., 2016). At a sufficient distance from the source of vibrations, the so-called Langefors (or also Koch) formula is used (in the general form)

$$V_{max} = K \,.\, Q^m \,.\, l^{-n} , \qquad (1)$$

where V_{max} - maximum ground velocity [mm.s^{-1}],
Q - weight of the charge [kg],
l - distance from blasting site [m],
K, m, and n are empirical parameters.

The empirical parameters are determined from the results of the experimental measurements, and they depend on the geological pattern of the area, on the distance between blast and structure, as well as on the method of blasting (for example, Spathis and Noy - eds, 2010, Kondela and Pandula, 2012). There are known locations where this relation shows a very high or conversely very low correlation coefficient (for example, Pandula and Jelšovská, 2008, Kaláb et al., 2013). Often, also in Czech Republic (CSN 73 0040), parameters m = 0.5 and n = 1 are recommended; the formula then takes the form:

$$V_{\max} = K \frac{\sqrt{Q}}{l} . \qquad (2)$$

The value of the K parameter (it depends on geological pattern and distance, it reflects attenuation of seismic waves) is calculated from the experimental measurements; the tables of the K values are often part of standards, and it can be used for the orientation evaluation. From this empirical relation, it is possible to estimate the weight of the charge so that the maximum values of the ground velocity do not exceed the limit ground velocity defined in the standard. These limit values are defined by the permissible load on the construction to take into account local geological pattern and classification of the evaluated structure.

Blasting operations produce seismic waves with a wide frequency spectrum, which depends on properties of the material being disintegrated, properties of the explosive and the blasting technique. The frequency spectrum in a seismic record of a blasting operation is further significantly affected by the environment which the waves propagate through; components of higher frequencies are more rapidly attenuated with the growing distance in the groundmass. Isaac (1991) introduced a reference chart of dependencies of' the frequency range of a seismic signal on the distance from the blasting point. It follows from the chart that, if we wish to have an undistorted seismic record, it is necessary for the frequency range of the seismic channel, especially in the case of small distances from the blasting point, to be as wide as possible (first of all as far as higher frequencies are concerned). The frequency spectrum of seismic signals induced by close-distance blasting in rock and semi-rock may contain frequencies up to 250 Hz. In addition, higher frequencies in the record may be affected by the resonance of the rock mass, the dimension of which is compared with the wavelength. All of these effects cause that the relationship mentioned above between the value of the maximum ground velocity, the weight of a partial charge and the distance can be determined only approximately, using statistical methods. The actual maximum vibration velocities have to be determined by monitoring (according to Kaláb et al., 2011). Many examples contained in technical literature show significant dispersion of the measured values.

To sum up, it is possible to state that the intensity of blasting induced vibrations depends on many parameters (for example, Kaláb, 2004, Spathis and Noy – eds, 2010, Pandula and Kondela, 2010), first of all on the manner of the vibration generation, vibration intensity (radiated vibration energy), the epicentral distance or the depth of the source, the structure of the groundmass which the seismic waves propagate through and the local geology in the location of the manifestation being monitored. The wide diversity causes affecting the value of seismic manifestation on the ground surface is the reason why it is impossible to obtain more credible results without a significant quantity of measurements and why the simple relationships cannot be derived, first of all for small distances. Mathematical modelling using various program systems is today an inseparable part of assessing the impact of technical seismicity on structures.

Examples of vibration effects on structures

The first example describes an analysis of vibration effect in a shallow mine - Jeroným Mine located in West Bohemia, the Czech Republic. Experimental geomechanical and seismological measurements are performed in the mine so that geomechanical stability of the whole underground complex may be evaluated (for example, Kaláb and Lednická, 2016; Knejzlík et al., 2011; Kaláb et al., 2010). Permanent seismological monitoring has been carried out since 2004 using a seismic station JER1, installed in the mine about 35 m below the surface in one of the largest chambers. From 2004 to 2006, the seismic station JER 1 monitored especially effects of blasting operations during the reconstruction of drainage adit. From 2008, more than 2000 earthquakes in West Bohemia were recorded at the station JER 1 during three intensive seismic swarms in 2008, 2011 and

2014. Other sources of vibrations, recorded at the JER1 station, represent quarry blasts from nearby quarries, and vibrations generated by traffic on the road situated above the mine. Detailed analysis of the vibration effect caused by individual seismic load sources is performed in time and frequency domain (for example, Lednická and Kaláb, 2013; Kaláb et al., 2015). Some fractures are observed also using glass markers (Fig. 7).

Fig. 7. A glass marker in a fracture in the underground space of the Jeroným Mine. (Photo: Lednická).

The Jeroným Mine is located at a distance of about 25 km southeast of Nový Kostel focal zone, where seismic activity occurs in the form of seismic swarms. Maximum vibration effect in the mine is caused by these earthquakes. Maximum measured velocity values reached up to 0.8 mm.s^{-1} for an earthquake with magnitude M_L = 3.6 (Fig. 8). Technical seismicity also causes vibration effect in underground spaces. It was documented that maximum velocity values and prevailing frequencies of records from individual sources change significantly. Maximum values from the nearest quarries Vítkov (Fig. 9) and Krásno are usually within the range of 10^{-3} – 10^{-2} mm.s^{-1} (not too significant for stability assessment). The prevailing frequency range of recorded waves is 1 – 6 Hz. The passage of vehicles produces some weak vibrations; component values of maximum velocity are up to 10^{-2} mm.s^{-1} measured at the seismic station JER1. The frequency range of prevailing waves is very narrow at 9 – 15 Hz. This information is important especially for stability assessment and numerical modelling of a seismic load of the underground spaces. Based on current results (for example, Kaláb et al., 2015) we can state that the Jeroným Mine, as the whole complex of underground spaces, should be stable from the viewpoint of damage caused by vibrations. No sudden changes of convergence measurement and measurement of movement along fractures were detected in places with continuous monitoring during the seismic swarms. It is necessary to add that weathering of rock massif is a more important question (Lednická and Kaláb, 2016).

Vibration effect was also measured in different parts of Jeroným Mine. During the 2011 seismic swarm in Nový Kostel, more than 200 earthquakes with local magnitude $M_L \leq 3.3$ were recorded in the underground spaces (Lednická and Kaláb, 2016a); one permanent and five temporary seismic stations were used for the measurement and data analysis. These stations were placed in the mine at the depth ranging from 24 m to 53 m. The lowermost station was selected as the reference station for the analysis of vibration effect changes in comparison with other stations. The peak ground velocity during the phase of S-wave was determined for each recorded earthquake at each station. According to obtained results, it was found that the vibration effect in the mine is decreasing with increasing depth. Vibration effect at a depth of 30 m is approximately two times higher than at a depth of 53 m for vertical and both horizontal components. The calculated peak ground velocity ratio was plotted depending on depth under the surface (Fig. 10). Ratios from all seismic stations correlate with depth except one station at a depth of 24 m. This station was located near the surface in complicated geological and geomechanical conditions. The discrepancy of vibration effect at this place can be connected probably with the jointed rock massif and collapsed overburden consisted of rock blocks. It was also found that the ratio is almost the same for the local magnitude ranging from the 0.9 to 3.3.

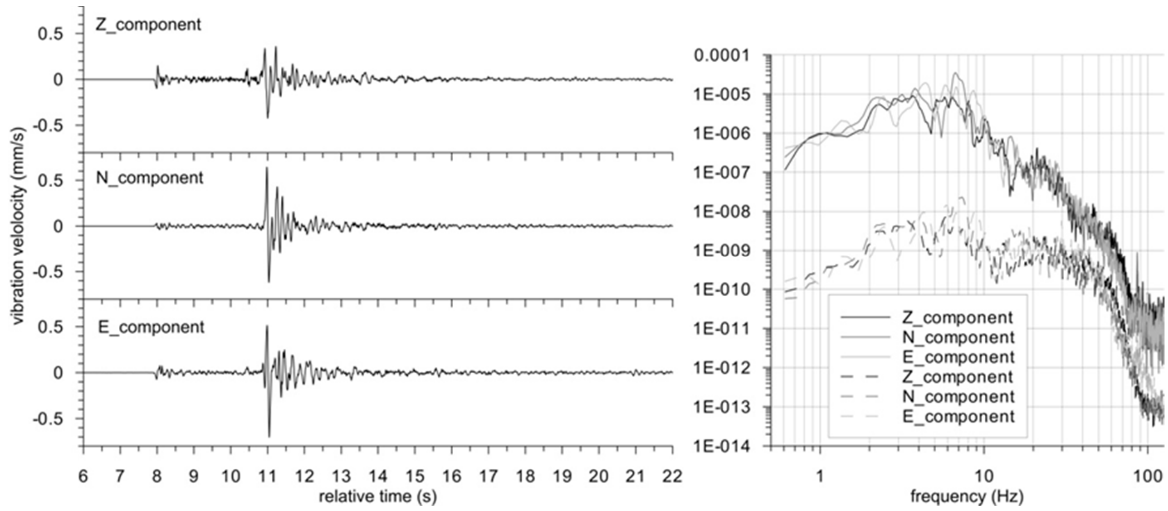

Fig. 8. Wave patterns and spectra of the earthquake from the Nový Kostel focal zone recorded at the seismic station JER1; left – wave pattern of M_L 3.6 earthquake on 4 August 2014; right – spectra of M_L 3.6 earthquake (solid line) and spectra of M_L 2.1 earthquake (dashed line).

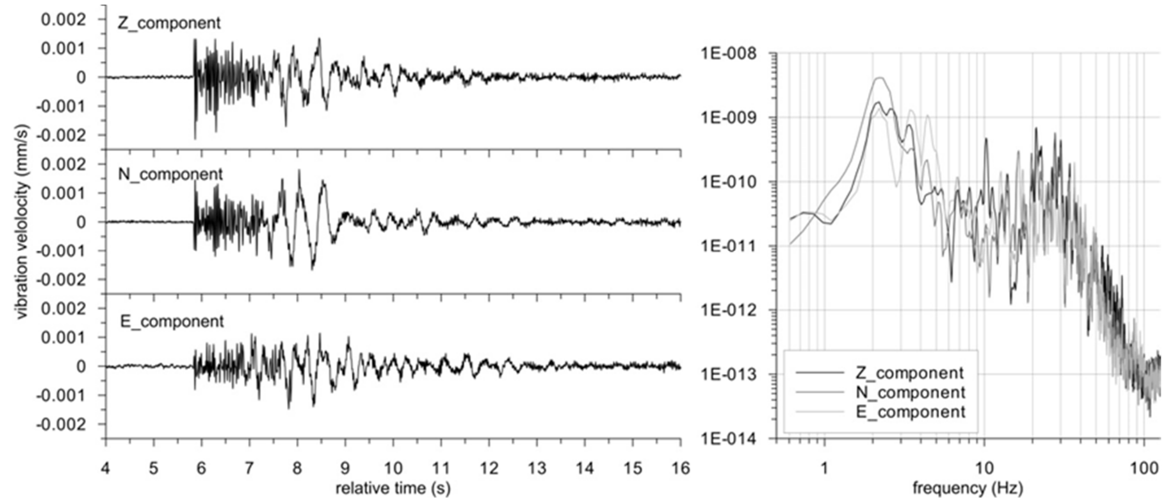

Fig. 9. Wave patterns and spectra of a quarry blast at Vítkov recorded at the seismic station JER1.

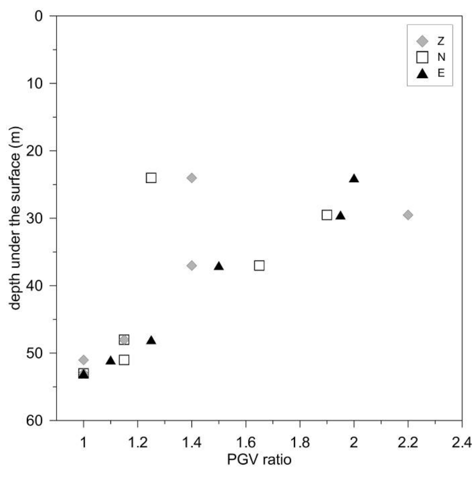

Fig. 10. PGV ratios depending on depth under the surface; depth of 53 m below the surface represents the reference depth for peak ground velocity ratio calculation (Lednická and Kaláb, 2016a).

Phyllite is excavated in the open pit mine near Jarnoltowek (Poland). This next experimental measurement documents resonant vibration generated on geotechnical structures – rock waste deposit that is located directly in the mined part of the quarry at a distance approximately 150 m from the quarry face. Blasting operations are used as mining technology so the rock waste dump might be influenced by these vibrations significantly. The first blast marked as BLAST1 was located on the nearest mined level, the second blast marked as BLAST2 was located on the higher level on the opposite side of the quarry (see Fig. 6); technological parameters of blasts were not published (Lednická and Kaláb, 2015). First presented seismic station (R) was also located in front of the dump on rock basement, second presented seismic station was located on given level of the embankment (D2 – twelve meters above basement). Obtained wave patterns are presented in Figure 10. Due to the short distances frequency range of seismic channel was set in the range 2 Hz – 200 Hz with sampling frequency 500 Hz; these values were sufficient for the measurement of blast-induced vibrations and the resonant vibration of the dump. Analysis of dump response in amplitude and frequency domains was performed by spectral ratio method and by detailed frequency analysis together with continuous wavelet (Morlet transform) time-frequency spectra method (for example, Lyubushin et al., 2014).

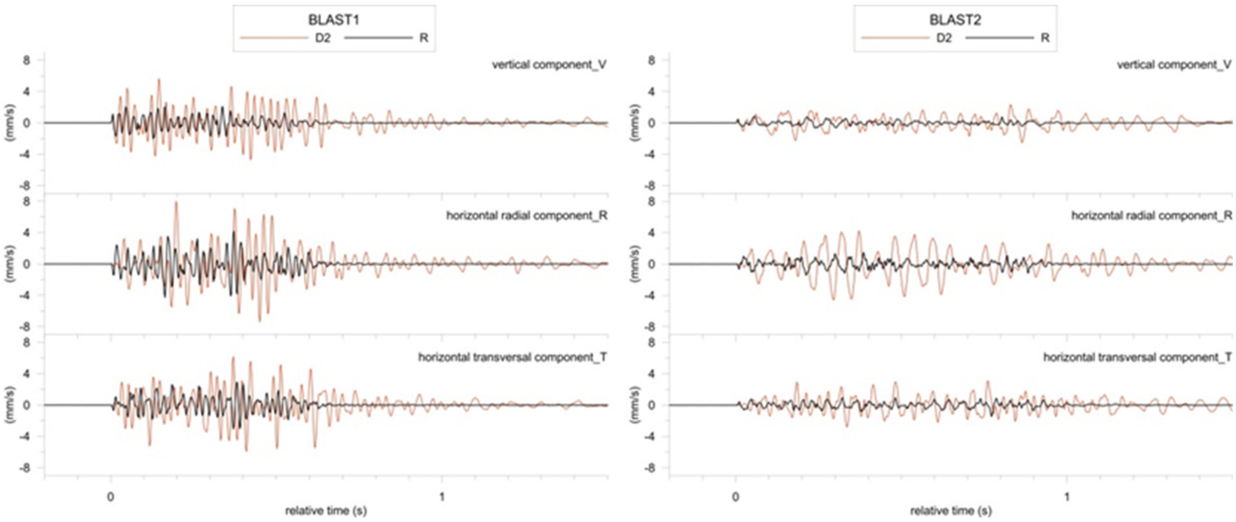

Fig. 10. Wave patterns of two blasts recorded on given seismic stations (R – rock basement, D2 – embankment). More details are described in the text.

The measurement was performed on four levels of the embankment of the rock waste (very fine material). Horizontal components of the ViGeo2 sensors were directed to radial (R) and transversal (T) directions; the third component was vertical (V). To obtain good contact of D1 – D4 sensors with the ground, they were placed into shallow holes. The maximum value of ground velocity during blasting reached 8 mm.s^{-1} on the radial component at the embankment´s height approximately 12 m. Spectral analysis proved a difference in the prevailing frequency range of two measured blasts, i.e. 40 - 50 Hz during the first blast and 15 - 25 Hz during the second one. It was also found that the prevailing frequency content of input ground motion probably influences the dynamic response of the dump, especially in the phase of the dump´s resonant vibration. Both analysis methods mentioned above proved that the resonant frequency determined at the different level of the embankment is decreasing with the increasing elevation of the embankment, which means that the thicker is the waste rock layer, the lower is the fundamental frequency. The maximum amplitude of resonant vibration was detected on the top of the embankment with the resonant frequency equal to approximately 4.0 Hz.

The last example is the evaluation of the influence of basin structures on the shape of wave patterns of induced seismic events. These vibrations are induced by blasting operations that are practised as a part of exploitation technology. A data set of interpreted wave patterns was collected from the data that were obtained during experimental measurement in the open pit Nástup Tušimice Mines in the North Bohemian Brown Coal Basin in 1996 and 1997 (for example, Kaláb and Knejzlík, 1999). Seismic stations were located both in the mine and in buildings the surrounding villages (an example of failure on buildings are in Figure 11).

Fig. 11. Two examples of cracks in buildings damaged by vibrations in the surrounding of open pit Nástup Tušimice Mines. (Photo: Kaláb).

The wave field that is generated after blasting operations is very complicated. The set of recorded wave patterns from positions with different geological conditions shows influences of basin structures on the shape of wave patterns. The wave patterns in this locality made in the distances in range 2 - 5 km are marked with the 3 - 7 s duration of the group of body waves. After that, the record of the group of surface waves manifesting themselves in a vibration of the harmonic type for 15 - 35 s follows in the wave patterns (example on Fig. 12). In this second group of waves with the prevailing frequency of about 2 Hz, the maximum recorded value of ground velocity occurs in the majority of records too. The obtained type of record in the discussed area is similar to some records of near shallow earthquakes. These records are also characterised by intensive surface waves whose amplitudes can exceed the amplitudes of body waves as well.

On the whole, in terms of evaluation of seismic load on buildings in surroundings of the discussed open pit mine according to the values defined in the Czech standard (CSN 73 0040) it can be stated that the value of 3 $mm.s^{-1}$, which is a minimum limit value at the "0" degree of damage (i.e., without any damage) was never reached during any blasting. If we admit the "1" degree of damage (i.e., the first damages – small failures), we can present that the minimum limit value of 8 $mm.s^{-1}$ is three times higher. The maximum recorded value of velocity amplitude in buildings was 2.8 mm $s^{-1,}$ but seismic loading is often repeated which then results in debasing of the technical conditions of the buildings.

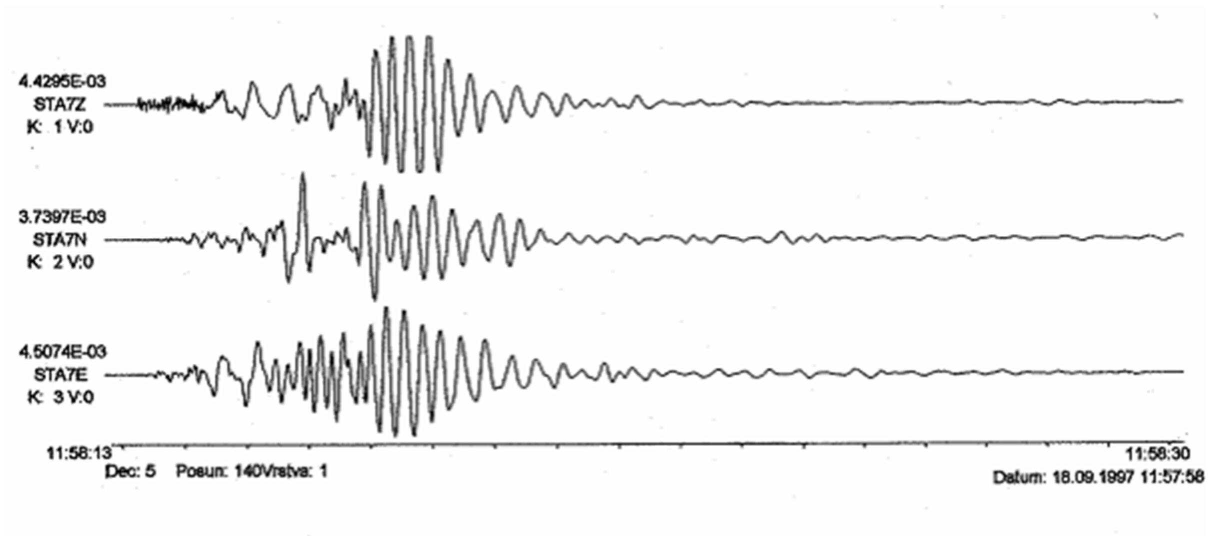

Fig. 12. Wave patterns of blasting operation recorded on concrete pillar located on the boundary of open pit Nástup Tušimice Mines (epicentre distance about 1.5 km).

Discussion

It is possible to find a large number of papers and books that described the main topic of this paper, i.e. influence of vibrations on structures (for example, Bolt, 1999, Towhata, 2008, Chudley and Greeno, 2014). Generally, three main parameters are necessary to take into account for comprehensive evaluation of this physical process: a source of vibration, the geological medium through which the seismic waves propagate, and local condition in evaluated point (geological and/or engineering problems). To obtain complete information, measurement system has to keep sufficient parameters, especially the frequency range of the whole seismic channel, sampling frequency, and proper anchorage of the sensor.

Methodology for evaluation of vibration on structures usually includes the following steps:

- Determination of acceptable load;
- Prognosis of the load;
- Determination of risk, eventually safe distance and other parameters;
- Description of failures including their photographs, with special view of historical and fissured structures;
- Measurement of seismic effects;
- Evaluation of safety for the obtained (from measurement) load, correction of the current state;
- Observing existing fissures and failures.

At present, permanent seismic monitoring with automatic data acquisition and primary interpretation of basic parameters is favoured in urban regions when significant vibrations are generated. Realisation of temporary seismic stations that will operate in suitable buildings during the whole period of seismic loading (generally weeks or first months) is supposed. Obtained results are at disposal to civil engineers, fire-fighters and also to custodians and occupants of influenced buildings. Usually, the web application is available with different access authority levels.

Lu (2014) defined major steps in the detailed analysis of ground-borne vibrations of building structures (engineering approach):

- Develop estimates of the force density. Can be based on previous measurements or a special test program. Adjustments for factors such as traffic speed, road surfaces, track support system, and vehicle suspension.
- Measure the point-source transfer mobility at representative sites. The transfer mobility is a function of both frequency and distance from the source. (mobility: velocity; inheritance: acceleration, displacement: receptance).
- Use numerical integration to estimate a line-source transfer mobility from the point-source transfer mobilities.
- Combine force density and line-source transfer mobility to project ground-surface vibration.
- Add adjustment factors to estimate the building response to the ground-surface vibration and to estimate the weighted sound level inside buildings.

It is also necessary to point out the second-hand influences of a seismic event on structures. The main discussed topics are influences on properties of the medium:

- Change of physical-mechanical behaviour of the subsoil of buildings;
- Change of stress conditions;
- Change of slope (downhill) stability.

Dynamic loading, even though its effect on underground structures is usually much smaller than that produced by the rock pressure, has been given greater attention of late, especially when dimensions of utility tunnels are designed. This type of load belongs among indirect loads, i.e., imposed deformation or limited deformation or constrained vibration. The problem of determining the magnitude of the affection of lining by dynamic loading is not simple to solve; the following methods are most frequently used:

- Recalculation from a wave pattern (usually a record of longitudinal and transverse waves) to tensile stresses and compressive stresses or shear stresses (Bulyčev, 1982); however, the complexity of the calculation together with the great number of constants and unknown quantities, makes this method impracticable.
- The use of calculation programs, which are capable of mathematical modelling of the dynamic action. The input parameters consist of basic characteristics of the dynamic action, for example, prevailing frequency of vibration, the maximum amplitude of ground velocity or acceleration, etc. Among such programs, we can name Plaxis, Cesar, ANSYS and other systems.
- The possibility of introducing a "dynamic coefficient" γ_a, which makes it possible to allow for the dynamic loading by means of adjustment of the value of gravitational acceleration (for example, Kaláb, 2007). The relationship between gravitational acceleration and the induced acceleration at the location of the structure, which is to be designed, is defined in the form of

$$\gamma_a = \frac{(a_d + g)}{g}, \qquad (3)$$

where a_d is dynamic acceleration [m.s^{-2}]
and g is gravitational acceleration [g = 9.80665 m.s^{-2}].

The latter method is based on the method of partial coefficients used in the ultimate load design concept. This principle is commonly used in Eurocodes. The method of partial coefficients is based on the verifying in all design situations whether the values for limit states are not exceeded if the design values are assumed in all design models to be applied to the loading, material properties, and geometrical data. The partial coefficients are partially based on the theory of reliability and partially on historical and empirical experience.

Problems of measurements and interpretation of vibrations originating at small distances during shallow tunnel excavation are paid great attention, first of all in settled areas (for example, Qui et al., 2008; Kaláb et al., 2011). Special attention is even devoted to the impact of vibration on vibration-sensitive devices.

Conclusion

Influences of earthquakes and quarry blasts on the structure are traditionally discussed and solved. Presently, vibrations generated by commercial explosives in tunnel construction may cause structural damage in urban areas. Therefore, suppressing the vibration effects and mitigating the possible hazard after blasting is important. The duration and also the number of explosives were carefully controlled. Urban tunnel construction induces not only changes in rock massif, but buildings and inhabitants in the nearest surroundings above a tunnel or underground working can be affected too. At present, permanent seismic monitoring with automatic data acquisition and primary interpretation of basic parameters is favoured in urban regions when significant vibrations are generated. Realisation of temporary seismic stations that will operate in suitable buildings during the whole period of seismic loading (generally weeks or first months) is supposed. Obtained results are at disposal to civil engineers, fire-fighters and also to custodians and occupants of influenced buildings. Usually, the web application is used with different access authority levels.

Generally, it is necessary to point out that disturbed objects, even without visible signs (for example, cracks), are more vulnerable. It is reflected by the resonance vibration of the smaller or larger building element and the acceleration of the "ageing" of the object. It is appropriate to include this fact in the assessment of a load of a structure by means of a coefficient (similar use as a construction reliability factor) if the discussed effect is proven by the passporting of the structure.

This paper summarises common information about the influences of vibrations on structures. The paper shows differences of vibration evaluation for earthquakes and technical events, esp. blasts. Examples of real wave patterns document common shapes (Fig. 8 and Fig. 9) and also signals with significant resonant vibrations (Fig. 10 and Fig. 12). Also, the wave field of high frequencies of seismic signal significantly interferes on the surface, and an important part of this field is also a response of the buildings' constructional elements.

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