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CAN INDUCED GROUND MOTION BE APPLIED AS A VIABLE STRUCTURAL LOAD IN SEISMIC ENGINEERING?

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HIGHLIGHTS

Seismic hazard from deep mining may be danger to buildings on the ground surface (see Fig. 1, 2, 3):

- newly designed structures on the ground should be protected at the design stage,
- for this purpuse Eurocode 8 is adapted using displacement approach,
- site dependent design response spectrum is constructed,
- a methodology linking forecasted ground velocity and design acceleration is derived.

STATEMENT OF THE PROBLEM

The induced seismic events with their m_L magnitudes from 3 to 4, or even exceeding 5, may generate substantial surface ground excitations. Shallow source, compare to natural earthquakes, together with small epicentral distances result in surface tremors, which may negatively influence environment, concern local communities and even lead to serious structural damages. For these reasons one may want to investigate what would be the level of these excitations danger for the safety of buildings and how to eventually mitigate these effects for existing buildings or for the planned buildings of the mine infrastructure in the vicinity of mines. Mitigation of natural earthquakes has led to special seismic codes used by civil engineers in structural design. Their application for the induced tremors is faced however with two important issues:

1) different spectral content and duration of the surface acceleration records of mine tremors compare to natural earthquake records,

2) different definition of seismic risk associated with induced seismic events compare to strong natural earthquakes expected with the return period of about 500 years at the place of the planned building site.

Particularly the second issue is difficult to tackle in a rational way. The philosophy of civil engineering seismic codes is directed towards extreme loads expected with 10% probability during 50 years of typical building exploitation. In terms of the seismic risk this means strong events with 475 years return period. In such cases the buildings should be designed including substantial non-elastic deformations and respective non-linear effects controlled by so called 'q' factors expressing plastic behavior of the designed structures.

On the other hand the extreme mine tremors may occur with return period of a few years which would require their different treatment than natural earthquakes in structural design. Still, however, the application of linear approach in structural response computations results in too conservative approach taking into account the surface Peak Ground Accelerations (PGA) often exceeding 30% g as recorded even during moderate rockbursts.


Fig. 1 Collapsed building in Welkom (South Africa) caused by mL 5.2 rockburst from December 8th 1976 (courtesy of dr A. Cichowicz, African Council for Geoscience).


Fig. 2 Damage to a building caused by mL 5.3 rockburst from March 9, 2005 in Stilfontein, South South Africa (courtesy of dr A. Cichowicz, South African Council for Geoscience).


Fig. 3 Damage to a gable wall caused by mL 4.3 rocburst from May 21, 2006 in Polkowice, Poland.

Type I

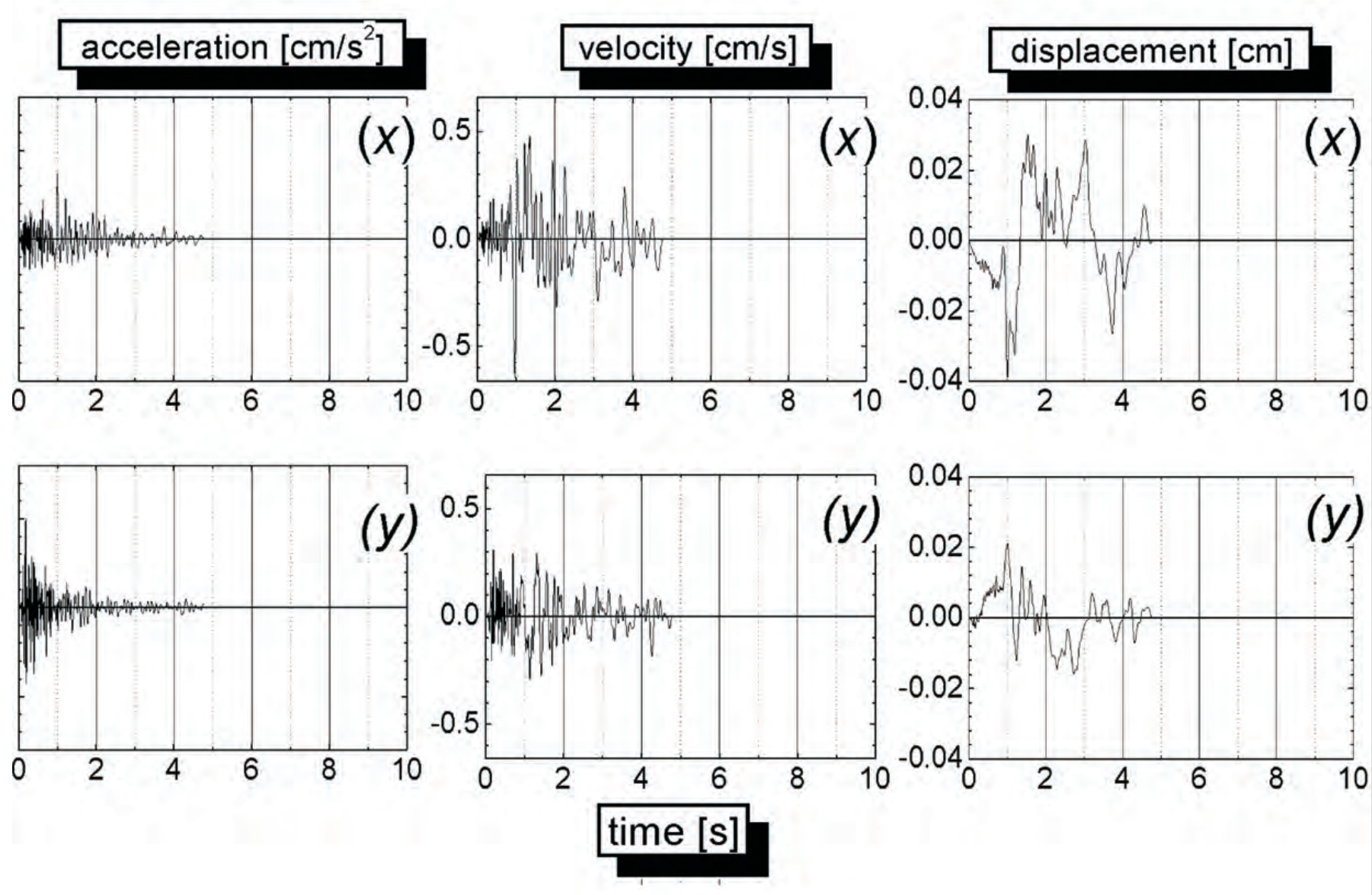

Fig. 4. Example of strong rockburst record of a type I event from February 2nd, 2001 (station *"3Maja"*) in Polkowice, Poland, (PGA_x=37cm/s², PGA_y=93cm/s², PGV_x=0.48cm/s, PGV_y=0.31cm/s)

Type II

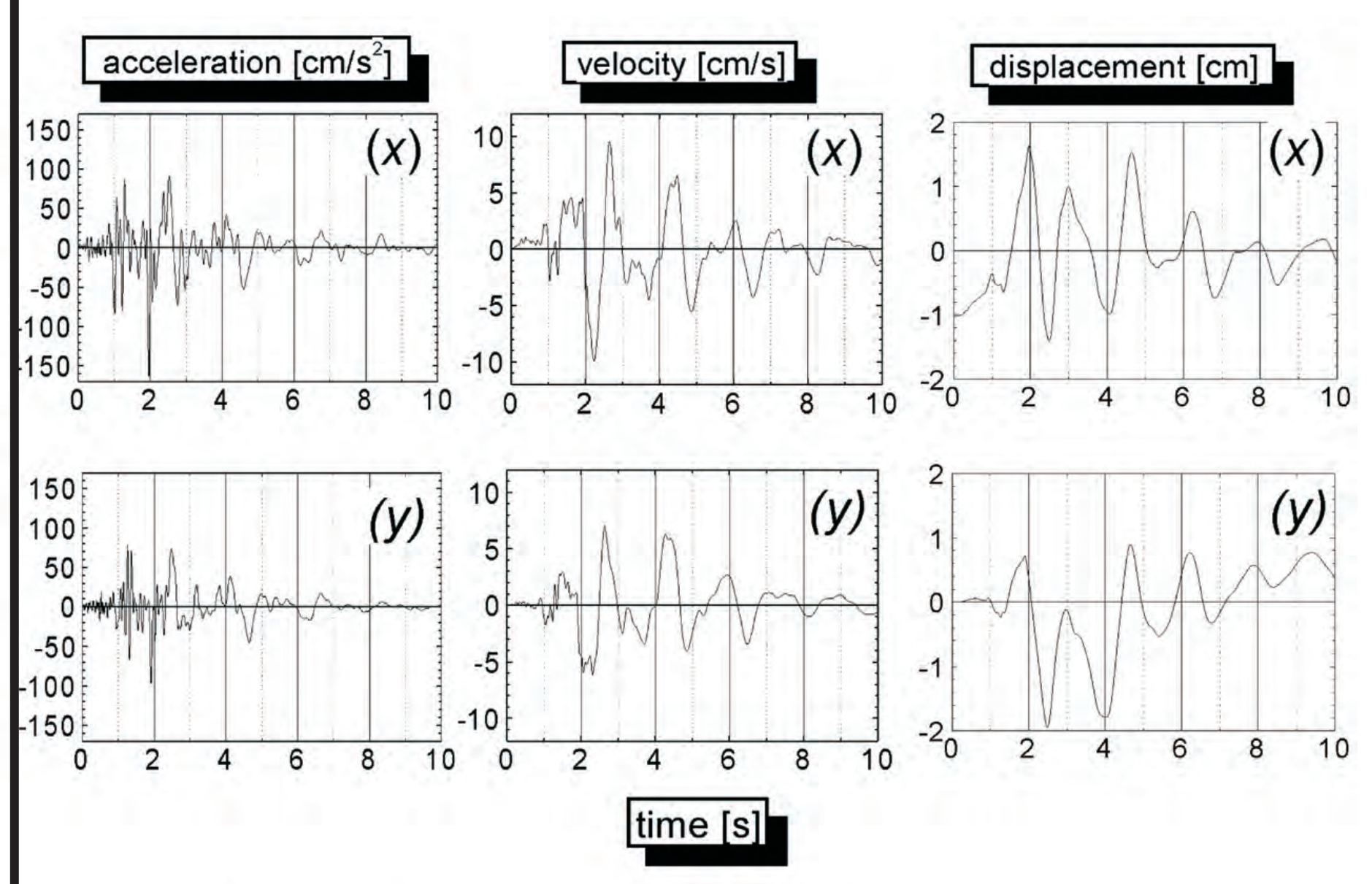

Fig. 5. Example of strong rockburst record of a type II event from February 20th, 2002 (station *"Miedziana"*) in Polkowice, Poland, (PGA_x=163.4 cm/s², PGA_y=96.5cm/s², PGV_x=9.95cm/s, PGV_y=7.04cm/s)

ROCKBURSTS OF TYPE I AND II

Two characteristic types of the surface records of the rockbursts were identified in 2004 in the paper by Zembaty (Rockburst induced ground motion - a comparative study, Int. Journal: *Soil Dynamics & Earthquake Engineering,* ELSEVIER, vol.24, pp. 11-23):

- Records of **type I with short duration (1–2 s) and Fourier spectra shifted to higher frequencies (about 20–40 Hz) similar to those from blasts** (Fig. 4). These records were collected from the events with low intensity and return period of 2–3 months.
- Records of **type II with longer duration (about 5s or more) and with dominating part of Fourier spectra below 5 Hz, similar to weak, shallow earthquakes** (Fig. 5). These records are collected from rather strong, rare events of return period 1-2 years.

CONSTRUCTION OF DESIGN RESPONSE SPECTRUM

The collection of intensive, type II ground recordings from the LGOM Copper Basin reached the number of 18 signals and this allowed to define a response spectrum representing rockburst seismic loading.

It is well known that ground motion at the surface is very strongly influenced by the geotechnical characteristics of the soil formations below the ground surface. Some seismic codes, including Eurocode 8 (EN 1998-1, 2005), International Building Code (IBC, 2009) and Italian Building Code (NTC, 2008), allow to account for site effects using a simplified method based on the introduction of a number of different soil categories to which specific, frequency-independent, soil factors are associated. These factors are used to modify the shape of the elastic acceleration response spectrum computed at a rocky site (reference spectrum). The parameter used to identify the soil category is VS,30, defined as a weighted average of the shear wave velocity in the uppermost 30 m of soil profile.

A purposely-developed procedure has been set-up to take into account the significant role played by local site conditions in the definition of the rockburst seismic action, including the litho-stratigraphic amplification effects in the rockburst design response spectra. The procedure was based on a stochastic approach to perform one-dimensional (1D) ground response analyses (Lai C.G., Corigliano M., Sánchez H.L. Some examples of 1D, fully stochastic site response analyses of soil deposits. *Proceedings of the ACES Workshop: Advances in Performance-based Earthquake Engineering.* Corfu, Greece; 2009). These fully stochastic site response analyses permited to account for the uncertainty of soil properties, as well as the variability of input motion. Three soil profiles were considered. As a result of this procedure elastic design response spectra were developed:

for soil profiles A,B:

$$S_a=a_g\beta(T)=a_gS\begin{cases}\left[1+\frac{T}{T_B}(2.5\eta-1)\right] & 0<T<T_B\\ 2.5\eta & T_B\le T\le T_C\\ 2.5\eta\frac{T_C}{T} & T_C<T\le T_D\\ 2.5\eta\frac{T_CT_D^2}{T^3} & T>T_D\end{cases}\quad(2)$$

for soil profile C:

$$S_a=a_g\beta(T)=a_gS\begin{cases}\left[1+\frac{T}{T_B}(2.5\eta-1)\right] & 0<T<T_B\\ 2.5\eta & T_B\le T\le T_C\\ 2.5\eta\frac{T_C^{1.5}}{T^{1.5}} & T_C<T\le T_D\\ 2.5\eta\frac{T_C^{1.5}T_D^{1.5}}{T^3} & T>T_D\end{cases}\quad(3)$$

η = damping correction factor

$$\eta=\sqrt{\frac{10}{5+100\xi}}\ge 0.55$$

the parameters depend on soil profile

S=0.8, T_B=0.1, T_C=0.85, T_D=1.3 for Eurocode 8 soil category A,

S=1.0, T_B=0.1, T_C=0.95, T_D=1.3 for Eurocode 8 soil category B,

S=1.5, T_B=0.3, T_C=0.80, T_D=1.3 for Eurocode 8 soil category C.

In Fig. 6 the plots of response spectra given by formulas (2) and (3) are presented

It should be noted that in traditional seismic design the linear response spectra, as in the formulas (2) and (3), are rarely applied as their application leads to too substantial internal design forces. Instead the response spectra accounting for inelastic response are applied with so called behavior factor 'q' changing from 1.5 to 5 effectively reducing the design forces. In case of extreme rockbursts which may appear with return period of 2-3 years such the approach is not possible. However some kind of "overstrength" effect should be taken into account to avoid too great internal forces. After careful analyses it was decided to allow application of q=1.5 and construct respective inelastic response spectra as follows:

for soil profiles A,B:

$$S_a=a_g\beta(T)=a_gS\begin{cases}\left[\frac{2}{3}+\frac{T}{T_B}\left(\frac{2.5}{q}-\frac{2}{3}\right)\right] & 0<T<T_B\\ \frac{2.5}{q} & T_B\le T\le T_C\\ \frac{2.5}{q}\frac{T_C}{T} & T_C<T\le T_D\\ \frac{2.5}{q}\frac{T_CT_D^2}{T^3} & T>T_D\end{cases}\quad(4)$$

for soil profile C:

$$S_a=a_g\beta(T)=a_gS\begin{cases}\left[\frac{2}{3}+\frac{T}{T_B}\left(\frac{2.5}{q}-\frac{2}{3}\right)\right] & 0<T<T_B\\ \frac{2.5}{q} & T_B\le T\le T_C\\ \frac{2.5}{q}\frac{T_C^{1.5}}{T^{1.5}} & T_C<T\le T_D\\ \frac{2.5}{q}\frac{T_C^{1.5}T_D^{1.5}}{T^3} & T>T_D\end{cases}\quad(5)$$

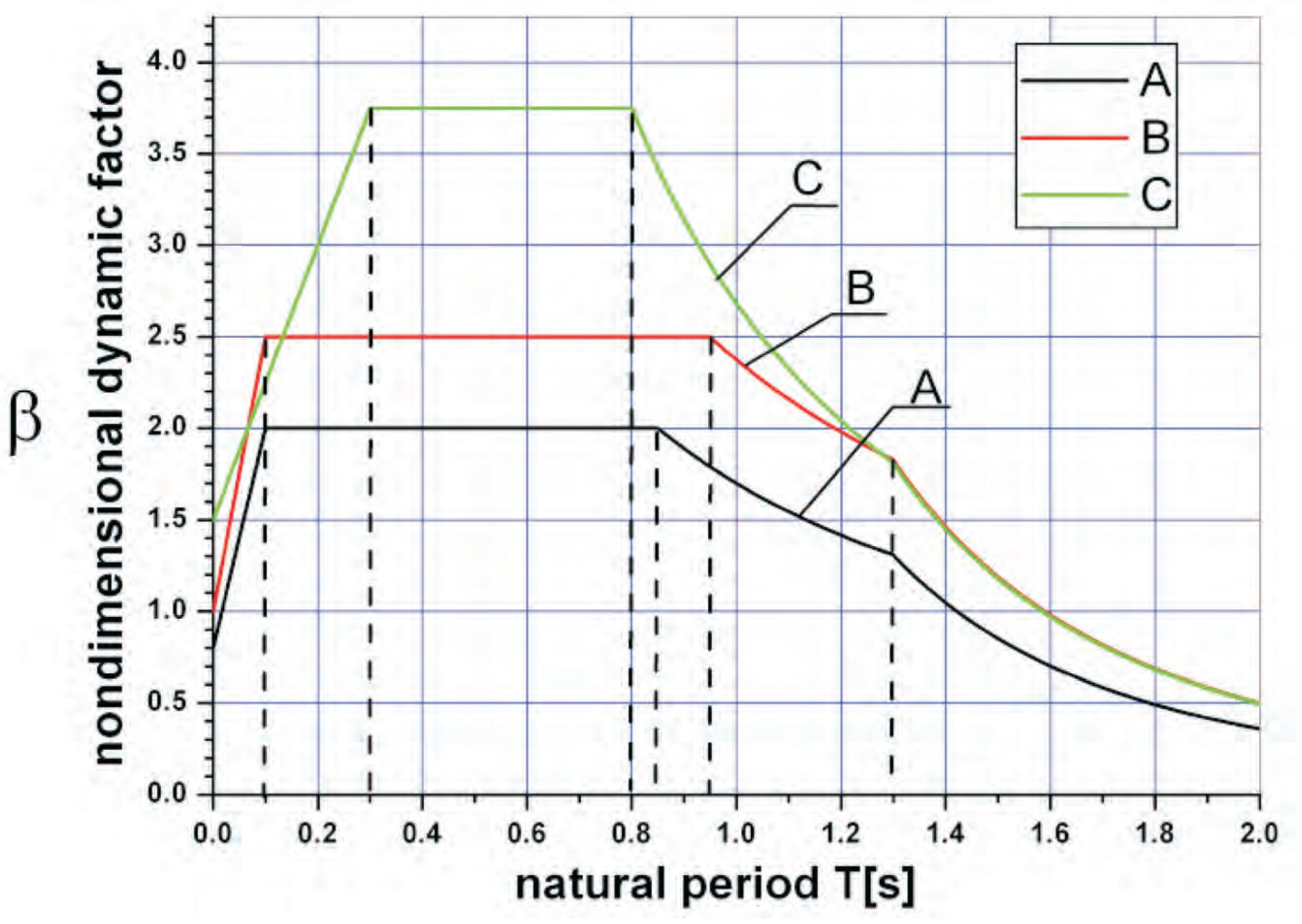

Fig. 6. Plots of elastic design response spectra for damping ratio ξ=0.05 and three Eurocode 8 soil categories A, B, C (design acceleration a_g=1m/s²)

SETTING THE DESIGN ACCELERATION VALUE

Difficult problem: decision about the design acceleration a_g must to be undertaken. For the mine tremors this is not easy. The solution applied in this case is based on the application of the displacement approach.

Consider Fig. 7, where displacement response spectra are shown for Eurocode 8 and two horizontal rockburst records along x and y axes. Respective horizontal peak particle velocity equaled in this case PGV_{hor}=6.37cm/s, where PGV_{hor} equals max$\sqrt{[V_x(t)^2+V_y(t)^2]}$.

The plots from Fig. 7 include the displacement response spectrum of the Eurocode 8 (straight line) calculated as $S_d=(2\pi/T)^2/S_a$. By changing design acceleration a_g the **EC-8 displacement response spectrum is set to best fit the response spectra of the two records in the range of typical natural frequencies of buildings,** here from zero to about 1.5s.

The best fit is reached for a_g=55cm/s². This means that the design acceleration 55cm/s² will lead to the relative displacements (deflections) in the structures the same as rockbursts with horizontal peak particle velocity of 6.37cm/s. This way one obtains an r factor between the rockburst intensity measured by PGV_{hor} and the design acceleration: $r=\frac{a_g}{PGV_{hor}}=\frac{55}{6.37}\cong 8.63$

The same fitting process was carried out more systematically, using minimization algorithms, between all the credible 18 records of strong rockbursts used to formulate response spectra (2-3). As a result r=5.77 was obtained with standard deviation 1.97. It was decided then that in the future designs in the LGOM region r=10 will be applied, i.e. $a_g=10v_g$, where v_g is the forecasted peak horizontal velocity for the surface of respective mining region. This means that accepting the peak horizontal velocity as the most appropriate measure of mine tremor intensity, a relation between the rocburst intensity at a site and the design acceleration was established.

The LGOM region was divided onto 4 mine regions:

- with v_g<1cm/s, where no measures against mine tremors are planned for the design
- with 2 cm/s> v_g > 1cm/s where only limited measures are planned,
- with 4 cm/s> v_g > 2cm/s where the response spectra (2-3) and (4-5) with a_g=40cm/s² should be applied
- with 6 cm/s> v_g > 2cm/s where the response spectra (2-3) and (4-5) with a_g=60cm/s² should be applied.

For all the four mining regions one can now rationally define a design procedure to mitigate rockburst effects in the designed structures.

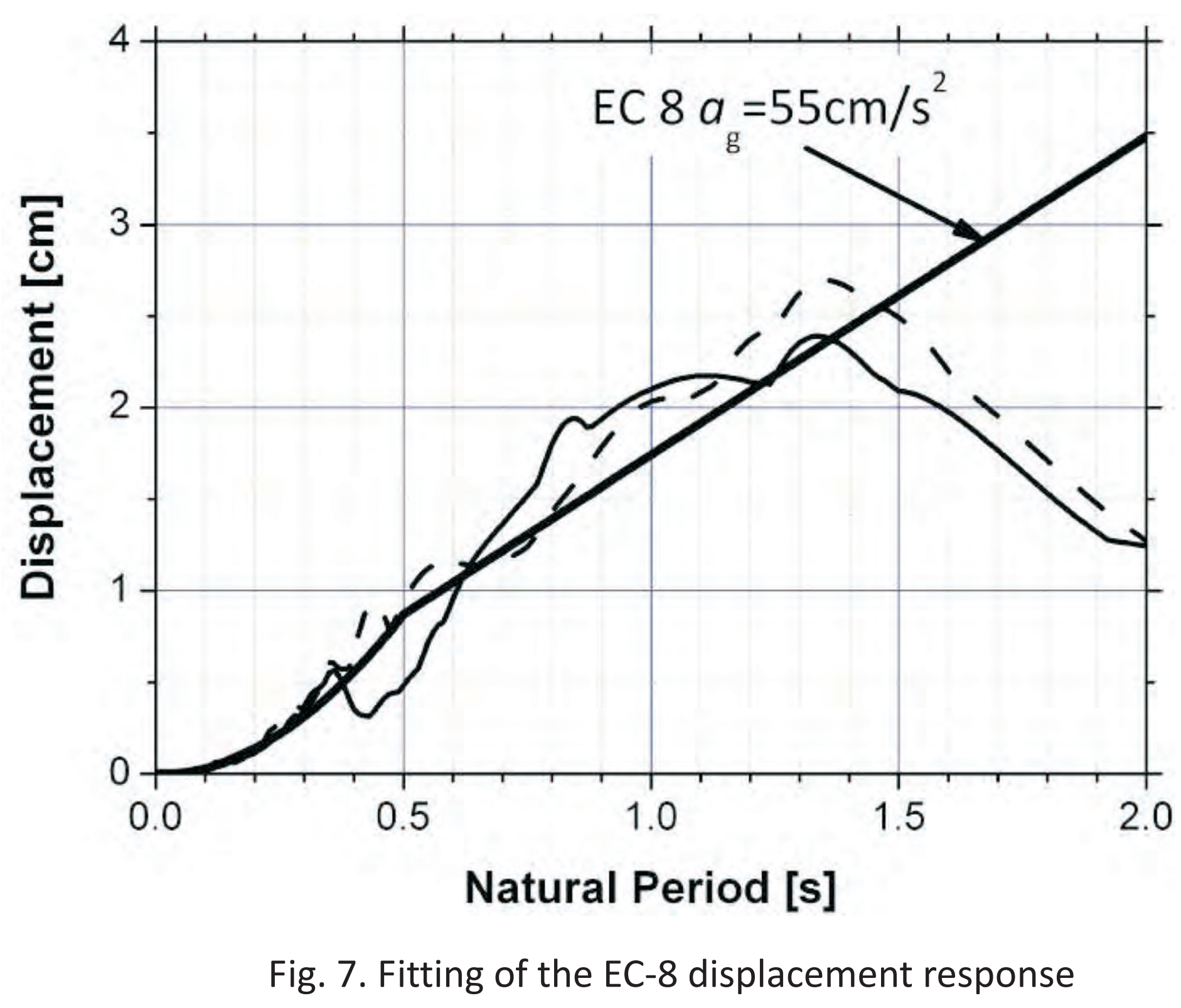

Fig. 7. Fitting of the EC-8 displacement response spectrum into rockburst displacement response spectra

CONCLUSION

A methodology how to apply Eurocode 8 methodology to design civil engineering structures under rockburst surface excitations is presented. Respective response spectra are obtained based on collection of strong rockburst records including the local site effects for three Eurocode 8 site profiles A, B and C. The region of mine induced seismic hazard is divided onto zones defined by surface horizontal peak particle velocity expected during the mine activity. The design acceleration a_g and respective response spectra define viable seismic load for Eurocode-8.