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Building damage due to vibration from rock blasting

K.M. Norén-Cosgriff [a,*], N. Ramstad [b], A. Neby [c], C. Madshus [a]

[a] Norwegian Geotechnical Institute, NGI, P.O. Box 3930 Ullevål Stadion, N-0806, Oslo, Norway
[b] Multiconsult, P.O. Box 265 Skøyen, N-0213, Oslo, Norway
[c] Statens Vegvesen - Norwegian Public Roads Administration, Norway

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ABSTRACT

Construction activities such as blasting, piling, compaction, excavations, and construction traffic can produce vibrations of sufficient strength to cause damage to neighbouring buildings and structures. Therefore, many countries have national limit values for construction vibration in standards. However, building damages assumed to originate from vibrations are seldom observed. This may indicate that today's limit values are unnecessarily strict. In this field little newer research has been undertaken to scientifically observe the onset of cracking, and there is a particular lack of information about which role the frequency content of the vibration plays. In this study the onset of blast induced cracking was observed in two instrumented test structures located in a rock quarry. Two buildings were constructed, one in cast-in-place concrete without reinforcement and one made of lightweight construction blocks in expanded clay aggregate (LECA). The buildings were instrumented with geophones and Fiber Bragg Grating Sensors (strain sensors). In addition, vibrations on the ground surface and air blast overpressure were measured. Test blasts were designed to produce increasing vibration values, starting with peak particle velocities (PPVs) around 20 mm/s and ending with PPVs above 250 mm/s. No visible cracks were found on any of the two buildings. However, the last blast, which produced PPVs above 260 mm/s, resulted in a residual displacement of 0.05 mm across the 110 mm strain gage length above the door of the concrete building. The results of the test indicate that the limit values of most national standards include a large safety margin for buildings founded on rock. Further, the dominant frequency was determined by different methods and the results show a considerable deviation, with a distinct difference between methods which determine the frequency in a short time interval around the highest peak and methods which are using the entire vibration time series. In addition, methods which determines the frequency in short time intervals show a large spread in the frequency between the different vibration cycles.

1. Introduction

Construction activities such as blasting, piling, compaction, excavation, and construction traffic can produce vibrations of such strength that they can cause damage to neighbouring buildings and structures. In many countries limit values for vibration from construction work are given in national standards. However, building damages assumed to originate from vibrations are seldom observed. This may indicate that today's limit values are unnecessarily strict. The determination of balanced limit values is very important because too strict limit values can delay the progress and increase the costs. A lot of research on how high vibration buildings can tolerate without damage was performed in the 50's - 70's, especially in Sweden and in North America. The limit values used in many countries today are based on these studies. However, the results were affected by the fact that instrumentation and analysis method at that time were less versatile and reliable compared to today's standard. Little newer research has been done, and there is particularly a lack of information about which role the frequency of the vibration plays. The Norwegian Standard is currently under revision to introduce a frequency filter that reflects the damage potential of vibrations with different frequencies and this information is therefore crucial. For this reason, an instrumented blast study was performed in Norway in November 2018, which is presented in this paper.

2. Earlier blasting studies

In 1957 a Swedish study of vibrations from short-range blasting was published, [1]. The data were obtained during a large reconstruction

* Corresponding author.
E-mail address: knc@ngi.no (K.M. Norén-Cosgriff).

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project in Stockholm. To reduce the excavation costs, the project decided to allow for blasts that could cause minor damages which could be repaired. This provided the opportunity to study the relationship between damage and ground vibrations. The results showed that no noticeable cracks could be expected to be found for PPV below 70 mm/s. All buildings in this study were founded on rock. In 1958 a Canadian study was performed of six buildings subjected to progressively closer blasting until damages occurred, [2]. The buildings, which were old but in good conditions, were either founded on a soft sand-clay, or on well-consolidated glacial till. In this test, charges were detonated progressively closer to the buildings until damage occurred. The study showed that damage was not likely to occur before the PPV was over 102 mm/s. The results from Ref. [1,2] and other previous studies conducted by the United State Bureau of Mines were compiled in Ref. [3,4], and a safe vibration limit of 51 mm/s was recommended. In the 70's the Bureau of Mines conducted a series of field studies of ground vibration and air blast damages, [5]. A total of 76 houses were monitored during production blasting performed in large surface coal mines, quarries and at construction sites. Threshold damages were reported down to PPV of 18 mm/s. However, all reported threshold damages were superficial cracking of the same type as caused by natural settlement, drying of building materials and variation in weather conditions. The results were compiled with results from earlier studies and an updated curve describing damage risk in relation to PPV was presented, which is still used as vibration limit values in the USA.

In the 70's - 80's, several comprehensive studies were conducted in Sweden. These studies were mainly reported in Swedish. The most important findings are translated and reproduced below. In Ref. [6] a large number of studies on ground vibrations and how they affect and possibly damage buildings are compiled. 91 buildings exposed to blasting vibrations between a few to over 100 mm/s were investigated. The investigations showed a probability for cosmetic damages of approximately 40% at a peak value of 50 mm/s. The investigations, however, were based on the difference between pre- and post-event inspection of properties. It is therefore likely that some of the reported damages could be conditions that were present before the blasts, but not discovered during the pre-examination. In Ref. [7] a study of building damages caused by vibration from blasting are described. A detached house made of lightweight expanded concrete blocks was used as a study object. The house was founded on good quality rock without any observed weaknesses. The house was exposed to blasting rounds with distances from about 100 m to just a few meters. The results showed that the critical vibration level in respect of damages was higher than PPV 90–110 mm/s. The damages that appeared clearly visible occurred at PPV $\geq$300 mm/s. Also [8] describes a study of building damages caused by vibration from blasting. The study object was a detached house founded on hard rock. The house had basement walls and floors in cast-in-place concrete, while the walls above ground were made of lightweight concrete with a brick cladding. PPV and frequency were measured from eight blasting rounds, all in quite short distances from the house (horizontal distance 1m-45 m). Vibration velocities up to PPV 1000 mm/s were registered during the study. The measurements showed no damages below PPV 110 mm/s and no major damages occurred until PPV 185 mm/s.

In recent years a comprehensive Indian study of building damages caused by mining blasting is presented in Ref. [9]. All together six test buildings were constructed at two sites close to opencast mines. The test buildings were monitored during production blasts starting at about 1800 m distance and working gradually closer until about 20 m distance. In a brick-mud-cement house, cosmetic cracks were detected at PPVs of about 50 mm/s, and in a two-story reinforced concrete and cement mortar building cosmetic cracks occurred at PPVs of about 70 mm/s measured at the first floor. In this test the frequencies of the blast vibration were less than 15 Hz for 94% of the recorded data. These low frequencies were believed to be a result of the soft top soil layer and the far-field monitoring locations. In Ref. [10] two case studies are presented on low rise buildings located nearby an excavation with blasting in rock. Both buildings were reinforced concrete structures with concrete block masonry partition walls. PPVs were measured on ground and compared to the US limit values. All measured PPVs were below 30 mm/s, which are lower than the limit value. Despite this, both buildings suffered threshold cracks and one of them even structural cracks. The distance between the buildings and the blasting was not reported, but it was stated that the blasts were nearby. This considered, the reported frequencies which were down to 5 Hz, are surprisingly low. An Australian field study of a single storey brick house located adjacent to a coal mine is reported in Ref. [11]. The house was instrumented with accelerometers and the building was monitored for cracks before and after each blast. In the monitoring period, which lasted over one year, the house was exposed to 43 blasts in distances from 50 m to 1 km. Measured PPVs on ground varied between 1.5 and 222 mm/s (peak vector sum) and the dominant frequency was in the range of 6 Hz–10 Hz. The study showed that opening and closing of cracks seemed to be more sensitive to rainfall rather than to vibration from blasting. No new damages from blasting were observed for PPV less than 75 mm/s. A Turkish study including field measurements and numerical study is presented in Ref. [12]. A five-storey reinforced concrete building was exposed to blasts from a quarry at about 750 m distance. The PPVs, measured on hard soil close to the building, were below 25 mm/s. The dominant frequency was as low as 5 Hz and hence the measured PPVs are above most limit values. Further, the fundamental natural frequencies of the building (1.7 Hz–3.2 Hz) were close to the dominant excitation frequencies of the blasts. Amplification that could give significantly higher vibration values higher up in the building was therefore likely. No damages were however reported.

3. Blast vibration standards

The Norwegian Standard NS 8141:2001 [13] gives guideline limit values to avoid damage to constructions from ground work. The guideline limits are values that buildings are supposed to withstand throughout repeated exposures without damages. For blast loading and residential buildings the guideline limit value for vibrations varies from a strict PPV of about 3 mm/s for a vibration-sensitive and brittle building on soft soil in long distance, to a PPV of about 80 mm/s for a building made of reinforced concrete founded directly on hard rock. The guideline limit values apply in vertical direction and shall be measured at or close to the foundation. They are calculated from a basis value and a set of factors which takes into consideration the ground condition, building category, type of foundation, building material, distance from building to vibration source and type of vibration source. The Norwegian Standard is currently under revision to replace some of these factors which indirectly takes the frequency content of the vibrations into account, i.e. the ground condition factor and the distance factor, with a frequency filter, that directly reflects the damage potential of vibrations with different frequencies.

The British Standard BS 7385-2 [14] gives guide values for building damage caused by vibration. The risk of vibration damage is evaluated, taking into account the magnitude, frequency and duration of the vibration with consideration of the type of building which is exposed. A frequency-based vibration criterion is given, which is judged to give a minimal risk of vibration induced damage. The guide value increases with increasing frequency. For transient vibrations, e.g. from blasts, the guide value for vibrations in the maximum of the three orthogonal directions at the base of the building is PPV = 15 mm/s at 4 Hz, which increases to 20 mm/s at 15 Hz and again to 50 mm/s at 40 Hz and above.

In Germany, DIN 4150–3:2016–12 [15] covers ground transmitted vibration and the effect on structures. The standard treats short-term and long-term vibrations separately. Short-term vibration typically covers blasting. The standard states that no damage due to vibration, which adversely will affect the serviceability of a structure, will occur if the guideline values of the standard are complied with. The evaluation of the structure is based on horizontal vibration measured in the topmost floor of the building. For residential buildings the guideline limit value for short term vibrations is PPV = 15 mm/s independent of frequency.

In USA the only federal regulation is issued by the Office of Surface Mining (OSM) [16]. However, the U.S. Bureau of Mines has established somewhat stricter vibration limit values which are often applied, even though not a federal regulation [5]. The limit values apply to vibrations measured on the ground in all three orthogonal directions. For buildings with dry walls, the vibration limit is 19 mm/s for frequencies between 4 Hz and 15 Hz, increasing to 51 mm/s at 40 Hz and above. For old buildings with plaster and lath walls, the vibration limit is 13 mm/s between 2.7 Hz and 10 Hz, increasing to 51 mm/s at 40 Hz and above.

4. Description of damage mechanisms

The energy released when a charge detonates performs for the most part useful work by breaking and moving rock. However, part of the energy produces wave movements in the surrounding ground. A blast initiates different types of ground waves which propagate with different speeds: relatively fast compression waves, shear waves with about half the speed of the compression waves (unless in saturated lose soils), and surface (mainly Rayleigh-type) waves with slightly slower speed than the shear waves. Rayleigh waves appear only down to a depth corresponding to about one wavelength, and decay therefore slower with distance than the other two wave types. Hence, Rayleigh waves dominate already at relatively short distances from the source.

Many studies have shown that the peak particle velocity (PPV) is the most relevant parameter in assessing blasting vibrations effect on structures, but also that the propagation speed in ground plays a role, since the shear and bending that building elements are exposed to are generally considered to be the starting point for the assessment of damage risks [1–3], and [4].

The solution of the wave equation can be written as:

$$y = A\sin(\omega t - kx) \tag{1}$$

where.

y is the particle motion.

A is the amplitude of the particle motion

ω is the angular frequency, $\omega = 2\pi f$

k is the wave number, $k = \frac{2\pi}{\lambda} = \frac{2\pi f}{c}$

c is the propagation speed of the wave type in question.

Considering the vertical particle motion, the dynamic shear strain in a building that flexes with the distortion of the ground surface can then be calculated as

$$\gamma = \frac{dy}{dx} = -kA_V\cos(\omega t - kx) = -\frac{V_V}{c}\cos(\omega t - kx) \tag{2}$$

where.

V_V is the amplitude of the vertical particle velocity.

Hence, the maximum shear strain will become

$$\gamma_{max} = \frac{V_V}{c} \tag{3}$$

The shear strain imposes a tensile strain at 45° angle to the shear strain

$$\varepsilon_{xy,max} = \frac{1}{2}\gamma_{max} = \frac{V_V}{2c} \tag{4}$$

Since surface waves usually have the lowest propagation speed and the highest amplitude, they will cause the highest shear strain and hence expose buildings to the greatest stresses. Rayleigh like surface waves involve particle motions both in horizontal and vertical directions that are of the same order of magnitude. The horizontal component will often be most important and impose the highest maximum tensile strains, which will be

$$\varepsilon_{tensile,max} = \frac{V_H}{c} \approx \frac{V_V}{c} \tag{5}$$

Eq. (5) considers tension and shearing of the building as the wave front passes. However, bending of the building together with the ground when the vibration wave passes can also be a possible damage mechanism. This is especially the case for soft ground conditions when the wave speed is low, and the wavelength of the surface wave can be in the same range as the length of the building.

The radius of curvature (R) for the vibration wave can be determined as

$$R = \frac{1}{\left|\frac{d^2y}{dx^2}\right|} \tag{6}$$

Assuming that the building deforms with the ground, the maximum strain in the building is found where the radius of curvature has its minimum. Eq. (1) and Eq. (6) give the minimum radius of curvature

$$R_{min} = \frac{1}{A_V k^2} = \frac{\lambda^2}{A_V(2\pi)^2} = \frac{c\lambda}{V_V 2\pi} \tag{7}$$

The maximum strain from bending in a building with a height H can then be estimated as

$$\varepsilon_{x,bend,max} = \frac{H}{R_{min}} = \frac{V_V 2\pi H}{c\lambda} \tag{8}$$

Another possible damage mechanism is amplification of the vibration velocity because of dynamic building response, which occurs when excitation and the building's natural frequencies are close. Buildings have natural frequencies in a wide frequency range, but those connected with the first modes are the most important since they have largest amplitude and cause the highest strains. According to Ref. [14], they are usually found in the frequency range from about 4 Hz to 15 Hz. In Ref. [17] the response of a reinforced concrete frame structure exposed to vibrations from underground blasts was studied numerically. The study showed that the low frequency global building modes dominate the response if the vibration frequency is low, while the first local modes for different building elements dominates for higher frequencies. Amplification factors in buildings were investigated in Ref. [5,9]. In Ref. [5] typical values of 1.5 for the entire structure as a whole, and 4.0 for individual panels or components at their respective natural frequencies were reported. Above 40 Hz the amplification factor was below 1.0 for all frame residential structures. In Ref. [9] maximum amplification factors between 2.6 and 5.2 were reported for normal structures in the frequency range between 2.5 and 24 Hz. In Ref. [11] amplification factors from less than 1.0 and up to 4.0 were reported for dominant excitation frequencies below 30 Hz. In addition to amplification, different building parts may vibrate out of phase, or move relative to each other, leading to cracking.

5. Cracking and critical strain for building components

Cracking occurs naturally over time in all buildings. Building materials expand and converge in connection with changes in the material's moisture content, temperature and creep, wind loads, and in some cases pre-stressing forces and settlements. If this movement is prevented, or if two different materials with different movements are joined, stresses that can lead to cracking occur [18]. According to Ref. [19] strain caused by moisture and temperature movements are large compared to strain caused by vibration. Vibrations usually do not produce strains that are above the critical strain of building materials. However, vibration-induced cracking may occur when the vibration induced strain combined with the pre-existing strain exceeds the critical strain of the material.

Tests on masonry and concrete were reported in Ref. [5]. The tests showed that poured concrete walls are much stronger than block walls and require high levels of strain to induce cracking, i.e. typically about 300 µstrain. Block walls on the other hand do not act as monolithic bodies, but strain concentrates at the joints leading to about 10 times higher strain levels across the joints than within the adjacent blocks. In Ref. [5] typical failure strains of the mortar joints were reported to be about the same as for concrete, i.e. about 300 µstrain. According to Ref. [18] the critical strain for most stone materials is about 100–200 µstrain, while it is much lower for the joints. However, it is not clear if this conclusion takes into consideration that the joints may be exposed to higher strain levels than the blocks if the building is exposed to a uniform shear deformation.

According to Ref. [14], the wall and ceiling material are often the most vibration sensitive parts of the building. In Ref. [5] results from several studies on strength of building materials were compiled. Old plaster and lath walls were shown to have lower critical strain than more modern gypsum wallboards with paper backing. Results from strength tests showed large variances, with typical values for tensile failure strain of gypsum wallboard of about 1000 µstrain. Assuming a stress concentration of 10 above doorways and windows, this corresponds to a uniform shear deformation that gives 100 µstrain.

6. Test site, test buildings and instrumentation

The test site for the present experiments was in Spulsåsen rock quarry in Våler municipality in Hedmark, Norway. A geological survey of the test site showed fine to medium-grained red granitic gneiss containing lenses of amphibolite, with dominating direction of foliations from the blasting area towards the buildings, Fig. 1. After the blasting tests were finished, core samples were taken from intact rock in front of the test buildings and tested in the laboratory. For the direction parallel to the foliations, the laboratory test showed an average velocity of 4260 m/s for compression waves, and 2644 m/s for shear waves, and an average density of 2646 kg/m^3 [20].

Two test buildings were erected at the test site, one in cast-in-place concrete and one with lightweight construction blocks made of expanded clay aggregate. The latter is further referred to as the LECA building. Fig. 2(a) shows the test buildings and the test area. Both buildings had one door opening and one window opening. The two test buildings were mirrored, so that the sides with door and window openings were facing each other. At the top of each buildings, joists were laid and filled with 4500 kg crushed rock to simulate the mass and ground pressure from a typical detached house on top of the lower story. The buildings were founded on an approximately 500 mm leveled and compacted layer of gravel, over rock. The dimensions of the buildings were 5 x 2 x 2.4 (l x w x h) meter. The concrete building had 200 mm thick concrete walls without reinforcement, on top of a 400 mm wide wall footing of reinforced concrete. The walls and footing were cast-in-place with strength class C30/37 concrete, which was allowed to cure for 30 days before the blast experiments were performed. The LECA building was made of 250 mm blocks with mortar in the joints and was coated with 15 mm plaster on the outside, Fig. 2(b). As cracks in wall- and floor tiles are a common reason for complaints from neighbors to blast sites, the inner wall of the LECA building that faced the blasting area was covered with tiles, Fig. 2(c). The LECA building was constructed in accordance with the supplier's instructions [21]. The building was founded on top of a wall footing made from 330 mm wide LECA foundation blocks with reinforcement. LECA U-blocks with reinforcement steel was used above the door and window opening and in the top row, Fig. 2(d).

Each building was instrumented with eight three-axial velocity sensors (geophones), Fig. 3(a), and eight dynamic strain sensors, Fig. 3 (b). Three velocity sensors were mounted close to the foundation (about 20 cm above), corresponding to typical sensor positions for measurements according to the Norwegian Standard. The other five velocity sensors were mounted up on the walls to detect possible amplification. Sensor positions were identical on the LECA and the concrete buildings respectively. In addition, vertical vibration in three positions on the ground surface and air blast pressure in two positions were measured, Fig. 3(c). One microphone for air blast pressure measurement was mounted on the LECA building on the side facing the blasting area, and one microphone was placed to the left of the LECA building in approximately free field conditions. The location of the buildings and all sensors were determined by GPS surveying.

For the vibration and air blast measurements the AVATrace M80 measurement system was used. Each three-axial sensor was connected to its own autonomous four channel logger. The fourth channel of each logger was used as a joint trigger channel, ensuring that all measurement

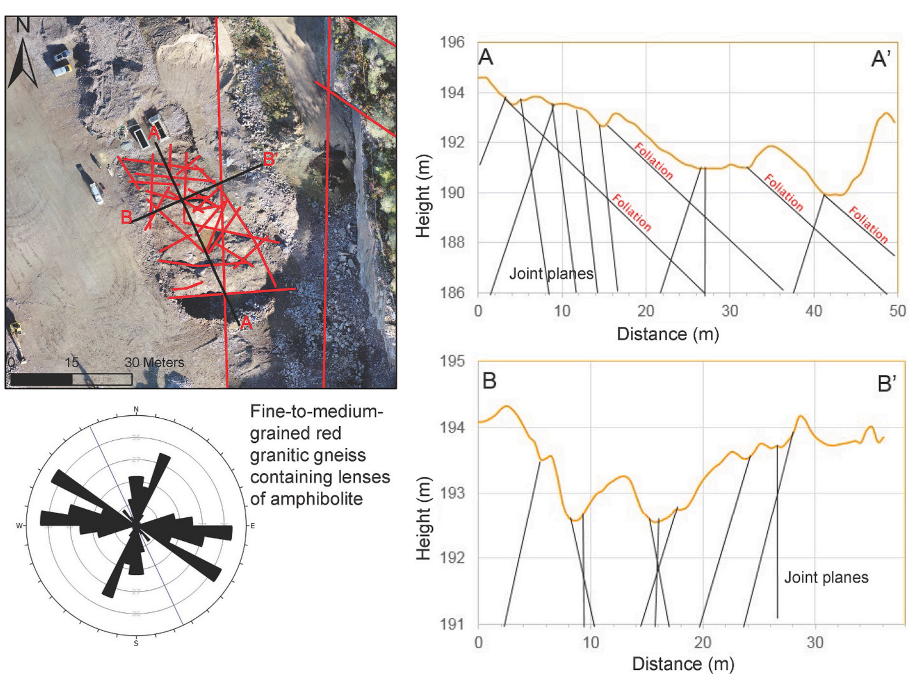

Fig. 1. Results from geological survey of test site.

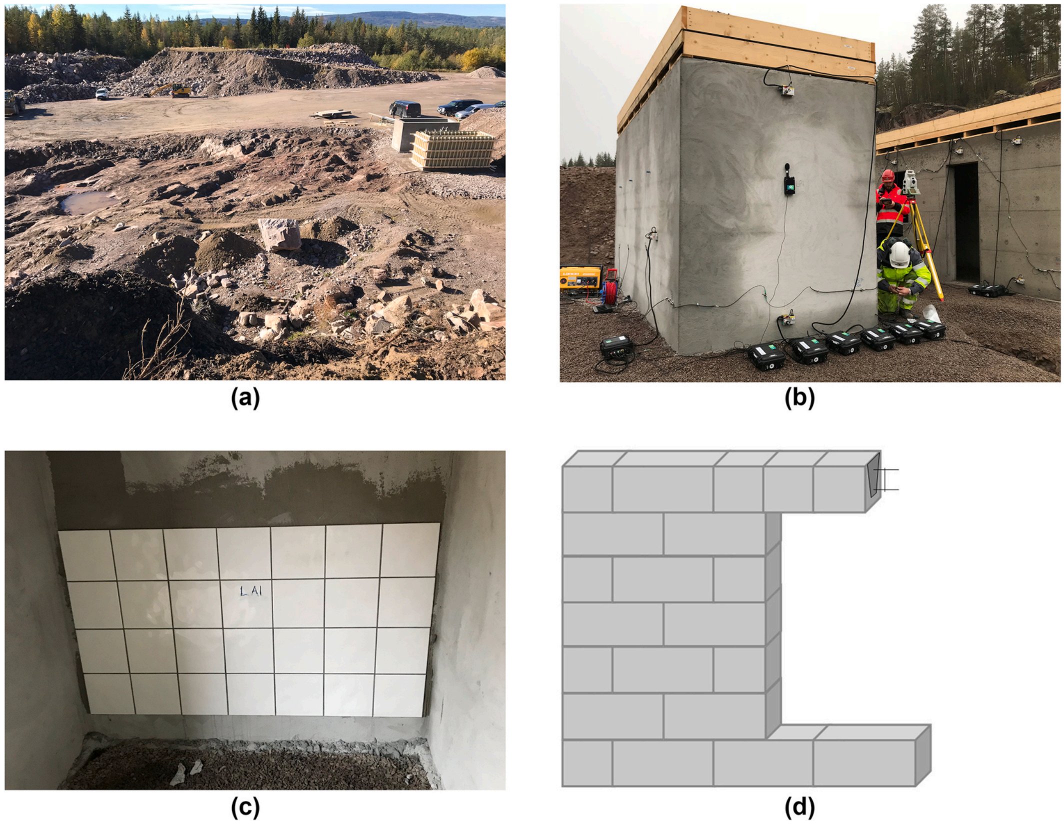

Fig. 2. a) Test area (left) and test buildings (right). The cast-in-place concrete building is under construction. b) Instrumented test buildings, LECA (left) and concrete (right). c) Tiles on the inside of the LECA building. c) U-blocks with reinforcement above window and door openings in the LECA building.

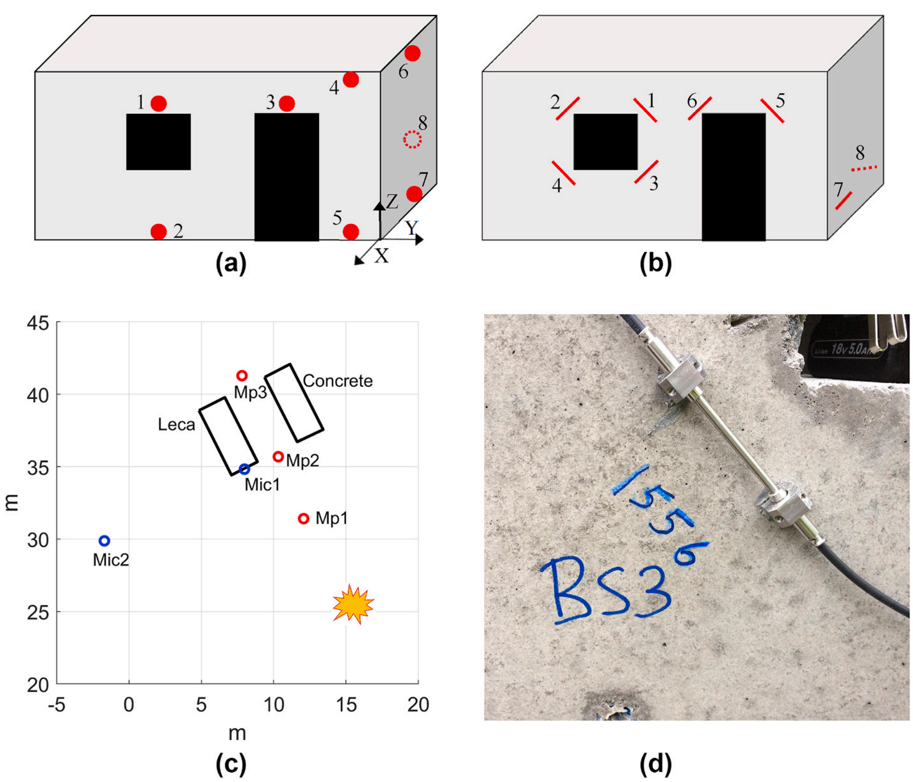

Fig. 3. Instrumentation. a) Position of three-axial geophones. b) Position of strain gauges. Dashed lines illustrate sensors on backside of building. c) Plan view: Position of vertical geophones on ground and air blast microphone. d) Strain sensor mounted below window on concrete building. The blasting area is to the right of (a) and (b).

channels on each building were mutually synchronized. The measurement system operated with a 6000 Hz sampling frequency. The strain measurements were performed with a fiber optic measurement system from Micron Optics, using os3510 Fiber Bragg Grating Sensors (FBGS) [22]. The strain sensors were attached via rigid brackets that were bolted to the structures. Dynamic strains were measured over the gage length, of 110 mm. The sensors were mounted in a 45° angle above door openings and above and below window openings, Fig. 3(d). In addition, two strain sensors were mounted close on the side wall and back wall about 40 cm above the foundation, Fig. 3(b). The sampling frequency of the strain measurement system was 1000 Hz. This was considered sufficient since the frequency content of blast vibration usually are well below 500 Hz and was also later confirmed from the vibration measurements. The sensors measurement range was ±2500 μm. The strain sensors were assembled in joint fiber cables, making them mutually synchronized. The collected time series from all sensors were analyzed in MatLab.

7. Execution of blast test

The blast test was performed from 6th to November 9, 2018. The weather in the measurement period was cloudy with periods with light precipitation. The temperature was never below 0 °C and the average temperature was between 5 °C and 6 °C.

Five blasts rounds were fired consisting of all together 143 charged holes. Drill diameters were 76 mm. The maximum borehole depth in each round was between 4.5 and 6 m. A 1.5 m crushed stone stemming was used in all holes. The distance between the rows (burden) was about 2.0 m, and the spacing between the holes was about 2.5 m. Coordinates for top of all boreholes were determined by GPS surveying and drone 3D-scanning before the test. Coordinates for the bottom of the holes were determined by use of a borehole deviation probe. The number of holes detonated in one blast round varied from two single holes up to 53 holes. The first four rounds were all shot by single hole initiation, with a delay between each hole in a row of 10 ms. In the fifth blast round the holes were shot two and two simultaneously, starting in the centre of the rows. The delay between the rows varied from 10 to 60 ms. The total amount of explosives detonated in one blast round varied from 3 to 404 kg, and the explosives detonated per delay varied from 3.0 to 37.8 kg. The blasts were designed to give equal dynamic loading on each of the two test structures, as well as increased vibration strength, starting at a low value for the longest distance and increasing progressively as the blasts came closer to the test structures. The first blasting round had a minimum distance of 29 m from the test structures while the last had a minimum distance of 7 m. To gain full control, repeatability and traceability of the blasts, packaged emulsion and NG-based explosives together with electronic detonators were used. Fig. 4 and Table 1 describes the test setup.

The buildings were visually inspected before and after each blast round to detect and document any damage. In addition, the results from the strain measurements were reviewed correspondingly to detect any changes not visible to the naked eye.

As a rule of thumb, the ground surface vibration response to a blast will be dominated by body waves if the angle between a surface normal

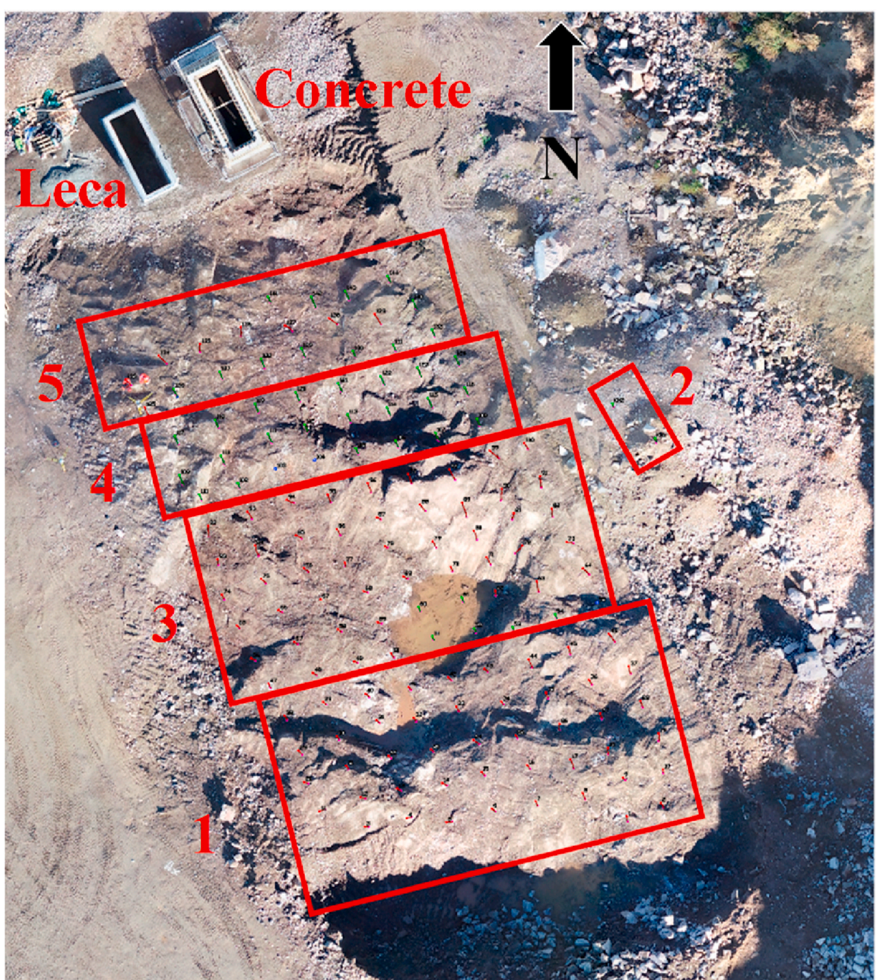

Fig. 4. Location of the blast rounds.

Table 1
Description of blasts.

Blast round	Date and time	No charged holes	Total charge (kg)	Max charge/delay (kg)	Min dist (m)		Min square root scaled dist (m/√kg)	
					LECA	Conc	LECA	Conc
1	6 Nov 14:47	46	222	8.4	28.9	30	11.5	11.2
2	6 Nov 15:36	2	6.5	3.5	26.5	23.5	14.5	12.9
3	7 Nov 14:35	53	404	14	17.5	18.5	5.4	5.7
4	8 Nov 12:56	22	287	16.4	12.3	13.2	3.2	3.3
5	9 Nov 14:17	20	266	37.8	7.4	7.2	1.0	1.1

through the receiver point and a line from the receiver to the source point (blast) is less than about 60°. For larger angles, i.e. larger distances, surface waves will dominate. The angles for all boreholes were calculated from the coordinate for the bottom of the boreholes and the coordinates for the closest parts of the buildings. This corresponds to the smallest possible angle, since the explosives are distributed in the boreholes and the angle will be larger for explosives closer to the surface. The calculated angles are tabulated in Table 2 and Fig. 5 shows the test area with the minimum angles for each blast round. The calculations show that surface waves can be assumed to dominate the response for blast round 1–3. For blast round 4 body waves may dominate for some holes in the closest row and for blast round 5 body waves may dominate for most of the boreholes.

8. Measured PPV, air blast pressure and strain

Table 3 shows measured PPVs and strain in the LECA building. For the LECA building, the measured maximum PPV varied from 32 mm/s to >260 mm/s, and the maximum peak strain from 72 μstrain to 733 μstrain, between the different blast rounds.

The highest PPVs were measured above the door opening (pos 3) and on the top of the long and short wall (pos 4 and pos 6), while the highest strains were detected above the door opening (pos 5 and pos 6). For most blast rounds vibrations in vertical direction dominated.

Table 4 shows measured PPVs and strain in the Concrete building. For the concrete building, the measured maximum PPV varied from 22 mm/s to >260 mm/s, and the maximum peak strain from 15 μstrain to >1750 μstrain. For most blast rounds vibrations in horizontal direction dominated. The highest PPVs were measured on the top of the short wall facing the blasting area (pos 6), while the highest strains, as for the LECA building, were measured above the door opening (pos 5 and 6).

The measured vibration and strain values were consistently higher in the LECA building than in the concrete building. The last and closest blast round was however an exception producing vibration values outside the measurement range of the recording system on both buildings, and very high strains on the concrete building. No visible damage was however found on any of the buildings during the visual inspections. Nevertheless, the closest blast produced a residual strain response above the door on the concrete building, which was not visible to the naked eye. The residual strain was measured to 500 μstrain over the 110 mm long sensor. If this differential movement was concentrated at one point, it would represent a 0.05 mm change.

According to the Norwegian Standard [13] measurements shall be performed in vertical direction at the foundation or on load carrying structure close to the foundation. Table 5 shows maximum PPV close to the foundation (maximum of pos 2, 5, 7). The vertical direction

Table 2
Minimum angle of incidence (deg).

Blast round	Angle of incidence (deg)
1	77–81
2	81–82
3	67–79
4	55–70
5	38–65

dominated for all blast rounds. The guideline limit value for both buildings calculated according to the Norwegian standard is 50 mm/s. The last three blast rounds produced vibration values above this guideline limit value.

Table 6 shows measured air blast pressure. Measured air blast pressure varied from 234 Pa to 750 Pa on the wall and between 119 Pa and 682 Pa in free field, excluding blast round two, which because of the relative positions of the blasts and microphones resulted in very low air blast pressures. The current Norwegian Standard does not include a limit value for air blast pressure. However, in connection with review of the standard a limit value of 500 Pa for the peak reflected pressure has been proposed. This is in accordance with the Swedish guideline limit value in, [23]. The measured air blast pressure from blast round five exceeded this value.

9. Strain calculated from measured PPVs

Strains were calculated from Eq. (5) and Eq. (8) by use of the average shear wave velocity in ground as determined from the laboratory tests, and vertical PPVs and frequencies measured on the buildings, close to the foundations. The calculated strains are shown in Table 7.

Blast round 5 was excluded from the calculations since all vertical velocity sensors were out of range. The maximum strain from shear, calculated according to Eq. (5) (excluding the last blast round), is 49 μstrain for the LECA building and 34 μstrain for the concrete building. The maximum strain from bending, calculated according to Eq. (8), is 12 μstrain for the LECA building and 10 μstrain for the concrete building. Comparison with measured values in Tables 3 and 4 show that the calculated total maximum strain values agree better with the measured values for the concrete building than for the LECA building. However, Eq. (5) and Eq. (8) estimate the strains in an intact, homogeneous wall. Strain (and stress) concentrates in the corners, and these strain concentrations may be large. The highest strains were measured above the door openings. Furthermore, the equations do not account for possible response amplification which is usually higher up in the building than close to the foundation. If the calculated strain values are compared to strain measured close to the foundation on the homogeneous side wall and back wall, the agreement is much better as shown in Table 7.

The strain calculations presented in Table 7 do not take the building response into account. To examine the possible effect of this, strain was also calculated from difference in vertical and horizontal vibration displacement, integrated from measured velocity in position 6 and 7 on the short side facing the blasting area (Fig. 3(a)).

Vertical strain, ε_z, and shear strain, γ, were estimated as

$$\varepsilon_z = \frac{\Delta\delta_{z,max}}{h} \tag{9}$$

$$\gamma = \frac{\Delta\delta_{y,max}}{h} \tag{10}$$

where.

$\Delta\delta_{z,max}$ is maximum difference in vertical displacement.

$\Delta\delta_{y,max}$ is maximum difference in horizontal displacement in y-direction

h is vertical spacing between sensor.

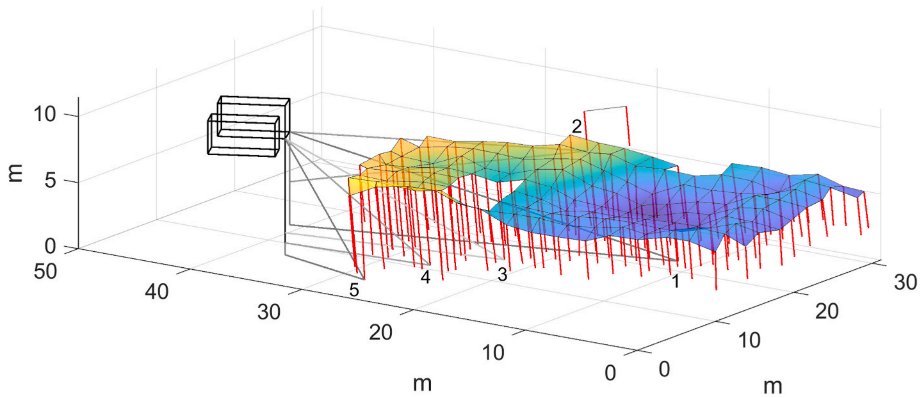

Fig. 5. Plot of test area with test buildings and boreholes (red lines). The minimum angle of incidence for vibration waves to the buildings are shown for each blast round. (For interpretation of the references to colour in this figure legend, the reader is referred to the Web version of this article.)

Table 3
LECA building. Measured maximum PPV (mm/s) and Peak strain (μstrain). Highest PPV in each blast round is marked in bold. See Fig. 3(a and b) for sensor location and measurement directions.

Blast Round	PPV X-dir	Pos	PPV Y-dir	Pos	PPV Z-dir	Pos	Peak strain	Pos
1	27	1	26	6	**32**	4	75	6
2	32	5	39	6	**52**	6	72	6
3	39	7	70	6	**89**	4	159	6
4	71	5	**133**	3	129	6	334	6
5	230	5	233	6	**>260**	all	733	5

Table 4
Concrete building measured maximum PPV (mm/s) and peak strain (μstrain). Highest PPV in each blast round is marked in bold.

Blast Round	PPV X-dir	Pos	PPV Y-dir	Pos	PPV Z-dir	Pos	Peak strain	Pos
1	**22**	1	15	6	14	7	17	6
2	17	3	21	6	**30**	6	15	5
3	46	3	**53**	6	45	4	24	6
4	79	3	**101**	6	81	7	40	5
5	**>260**	4	**>260**	3,4,6	**>260**	all	>1750	5

Eq. (4) and Eq. (10) give the tensile strain from shearing

$$\varepsilon_{zy} = \frac{\Delta\delta_{y,max}}{2h} \tag{11}$$

Table 8 shows calculated strain from difference in displacement. Calculated strain in vertical direction is from 2 to 7 times higher in the LECA building compared to the concrete building. There are many reasons for this difference. Among them are the jointing between LECA blocks, and that the LECA blocks have about ten times lower Young's modulus compared to cast-in-place concrete strength class C30/37 and will therefore react with larger strains when exposed to the same vibration load acting on the foundation. Strain from shearing calculated from difference in displacement, is higher than the values calculated from Eq (5) as shown in Table 7. However, the values calculated from difference in displacement (Table 8) are total values which also include effect of possible building amplification and bending. Further, the strain values in Table 7 must be considered rough estimates since they are calculated from the wave propagation speed, which is associated with uncertainty.

10. Determination of vibration frequency

One of the main challenges when assessing blast vibrations, is to determine the frequency content of the vibrations. Measurement

Table 5
Measured maximum PPV close to the foundation, max of pos 2, 5, 7 (mm/s).

Blast Round	LECA, PPV X/Y-dir	LECA, PPV Z-dir	Concrete, PPV X/Y-dir	Concrete, PPV Z-dir
1	21	30	13	14
2	32	48	14	29
3	43	86	29	43
4	86	119	64	81
5	230	264	197	266

Table 6
Measured peak air blast pressure (Pa).

Blast round	On wall, pos 1	Free field, pos 2
1	234	119
2	0.8	0.8
3	339	233
4	425	349
5	750	682

systems used for blast vibration usually determine the zero-crossing frequency by assuming that the time between the zero-crossing before a peak and the zero-crossing after a peak corresponds to half a period of the dominant frequency. Some systems only deliver the frequency for the maximum peak in the time series, while others like the AVA system used in the present study determine the frequency around each peak in the time series. However, there are also other methods for determination of the frequency content. In this study we have determined the frequency content by use of the response spectrum, instantaneous frequency computed from the Hilbert transform and characteristic frequency calculated from the power spectrum derived from the Fast Fourier Transform (FFT).

The response spectrum which are often used in earthquake engineering is calculated from the peak responses of a series of SDOF systems with varying natural frequency, that are forced into motion by the vibration time series to assess. The instantaneous frequency is computed as the derivative of the phase of the analytic signal found by using the Hilbert transform of the vibration time series. The characteristic frequency, f_{ch}, is calculated from the Power Spectrum using a maximum-likelihood approach as described in Ref. [24]:

$$f_{ch} = \sqrt{\frac{m_2}{m_0}} \quad m_n = \int_0^{f_{max}} f^n S(f) df \tag{12}$$

where.

S is the single sided power spectrum

Table 7
Calculated peak strain from shear Eq. (5) and bending Eq. (8) and measured maximum peak strain close to foundation on intact wall, pos 7/8 (μstrain).

Blast round	Shear		Bending		Total vector sum		Measured	
	LECA	Conc.	LECA	Conc.	LECA	Conc.	LECA	Conc.
1	13	6	7	3	15	7	14	10
2	20	12	8	4	22	13	22	7
3	36	18	15	7	39	19	26	8
4	49	34	14	11	51	36	41	14
5							342	41

Table 8
Peak strain calculated from difference in displacement measured on top of wall and close to foundation.

Blast round	LECA ε_z (μstrain)	ε_{zy} (μstrain)	Concrete ε_z (μstrain)	ε_{zy} (μstrain)
1	3	5	1	7
2	6	11	1	6
3	28	42	17	23
4	143	98	21	53
5		200		228

f is the frequency. The upper frequency is here restricted to 300 Hz in accordance with the measurement range prescribed in Ref. [13].

The second blast round contained only two single charges, delayed by about 3 s in between. The frequencies determined from these two blasts are therefore unaffected by the interaction between vibration contributions from the different detonations. The blasts were located about 30 m from the measurement position Mp3 on ground. The two detonations in this blast round were constricted by the surrounding rock (no free surface to break against). Table 9 and Fig. 6 shows the vibration frequencies determined from measurements in Mp3 on ground for the second blast round by use of the following methods: zero crossing frequencies determined by the measurement system, peak in response spectra, instantaneous frequency determined by the Hilbert transform and characteristic frequency of the power spectrum.

The frequency determined by the different methods deviates considerably, with a distinct difference between the methods which determines the frequency in a short time interval around the highest peak (zero crossing and Hilbert transform) and the methods which are using the entire time series (Response spectra and characteristic frequency of Power Spectrum). However, Fig. 6 indicates that the main frequency content is between 50 Hz and 130 Hz. This is lower than reported in Ref. [25] and what can be assumed from the ground condition factors in the present Norwegian Standard. However, relatively low frequencies measured on good rock has also been seen in earlier Scandinavian studies. In Ref. [26] dominating frequencies between 30 Hz and 120 Hz were reported on good rock in distances below 30 m. Ref. [8] reported frequencies between 40 Hz and 125 Hz on hard rock in distances from 17 m to 49 m, with the lowest frequencies for the shortest distances. In Ref. [7] frequencies between 45 Hz and 105 Hz were measured on good rock in distances between 36 m and 50 m. An explanation to those findings may be that the vibration characteristic close to the blast is more affected by factors of blast design, while at larger distances the transmitting medium of rock and soil overburden dominates the response, as described in Ref. [5].

Table 9
Vibration frequencies determined from measurements in Mp3 on ground for the second blast round by use of different methods.

Method	Frequency (Hz)
Instrument frequency (zero crossing around highest peak)	81
Instantaneous frequency from Hilbert transform	82
Peak in response spectrum	114
Characteristic frequency of Power spectrum	101

For the blast rounds that involve several holes with a delay between each hole, the measured time series become more complex, and it is more difficult to describe the frequency with a single number. Fig. 7 shows the measured time series, the corresponding response spectrum and power spectrum, and the frequencies determined by the measurement system for the third blast round. A comparison between Figs. 6 and 7 shows how the chosen delay interval, 10 ms, clearly affects the frequency by introducing a strong frequency component around 100 Hz for blast round 3. Further, for the third blast round the frequency determined by the instrument around the maximum peak value deviates considerably from the frequency range where most of the vibration cycles are located, Fig. 7(b). In these circumstances, the use of a single frequency value gives a poor description of the frequency content and demonstrates the difficulty of using frequency dependent limit values, such as in the British and American Standards. This is because a frequency dependent limit value requires that all frequencies with corresponding amplitudes from the blast are determined and compared to the limit value curve, and not just the frequency of the cycle with highest peak value, as many instruments provide as the only output. An alternative approach would be to implement a frequency weighting filter that directly considers the damage potential at different frequencies. This is the approach used in the ongoing revision of the Norwegian Standard.

Table 10 shows the vibration frequencies determined from measurements on ground (Mp3) and on the two buildings close to their foundations (pos 7 in vertical direction) for the different blast rounds by use of the zero-crossing frequency determined by the measurement instrument and the characteristic frequency calculated from the power spectra. Table 10 shows an apparent reduction of the dominant frequency with decreasing distance, which is not in accordance with the assumptions behind the distance factor in the Norwegian Standard and elsewhere reported in literature. However, if both the frequency and the distance are scaled with the charge weight as described in Refs. [27,29], this finding can be explained by the fact that the charges size was increased at the same time as the distance was reduced, which is usually not the case. Further, measured dominant frequencies on the two buildings are quite consistent despite the different building materials. This indicates that the dominant frequencies measured on the buildings are determined more by the blast design, the distance and by the transmitting medium between the blast and the buildings, than by the building material.

11. Building natural frequencies and amplification

As discussed in section 4 excitation close to the building's natural frequencies can cause high strains because of amplifications of the vibrations and that different building parts can vibrate out of phase or move relative to each other. The fundamental frequencies of the test buildings were determined by hammer excitation and from the measured building response in the second blast round (single hole), see Table 11 and Fig. 8. The results from the hammer excitation and blast excitation are rather consistent, but with slightly higher values for the hammer excitation in the Y-direction, in which the buildings are stiffest. This is probably because the hammer excitation failed to excite the entire structure in this direction. The fundamental frequencies for the test buildings are higher than reported in Ref. [5]. This can be explained

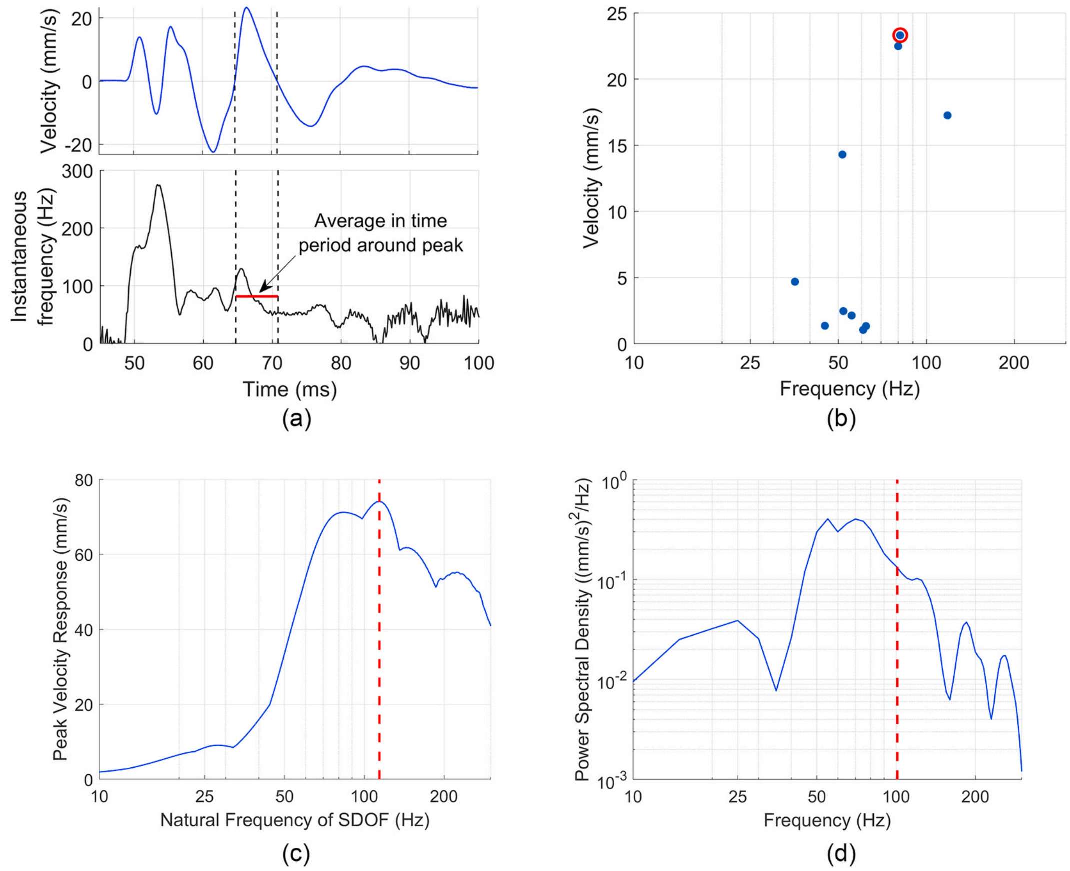

Fig. 6. Measured vibration velocity on ground (Mp3) from the second blast round. a) Upper panel: Time series. Lower panel: Instantaneous frequency determined from Hilbert transform. b) PPV vs zero-crossing frequency determined by the measurement instrument. c) Response spectrum. d) Power Spectral Density. The frequencies reported in Table 9 are marked with a red circle in (b) and red dotted lines in (a), (c) and (d). (For interpretation of the references to colour in this figure legend, the reader is referred to the Web version of this article.)

by the fact that the test buildings' dimensions were reduced compared to normal buildings.

Blast vibrations are more of a random than a deterministic process involving a lot of factors influencing the results in many unknown ways. For such processes statistical procedures are best suited to define descriptive properties of the data [28]. Here we have calculated the building amplification based on comparison of the power spectral densities (PSD) to get more stable results than obtained by comparison of the time series. The building amplification from ground to building and from a position close to the building's foundation to the building are calculated as

$$F_{B-G} = \sqrt{\frac{\sum PSD_B}{\sum PSD_G}} \tag{13}$$

where.

PSD_B is the PSD of measured velocity on the buildings in the position and direction of maximum value.

PSD_G is the PSD of measured vertical velocity on ground (Mp3) or close to the building's foundation (Pos 7) in the direction of maximum values.

The determined amplification factors are shown in Table 12. The amplification factors are in accordance with [5], which reported little or no amplification in structures from ground motions above 45 Hz. However, the amplification factors are lower than reported in Refs. [9, 11], and there does not appear to be any correlation between the dominant frequency (Table 10) and the amplification factor, which was the case in Ref. [9]. This can be explained by the fact that the dominant frequencies are considerably higher than in Ref. [9], where large charges and long distances gave rise to frequencies in the range of typical buildings fundamental natural frequencies.

12. Discussion

The performed blast test produced vibration values in the test buildings well above the current guideline limit values used in most countries. Despite this, no visible damage was found in any of the buildings. The buildings were however exposed to strain levels which were above critical strain levels reported in earlier studies. This may indicate that these newly erected constructions may tolerate higher strain levels than what has been found to cause cracking in other studies. Cured, but still young and flexible concrete and mortar, may get more brittle during further curing. In addition, drying makes permanent tension stresses develop over time.

The present study was designed to investigate damages to outer walls. Inner division walls and ceilings may be more vibration sensitive parts of the building, especially old plaster and lath walls. This needs to be further considered before the limit values in standards eventually are adjusted.

The relatively high fundamental natural frequencies of the test buildings compared to more common buildings, may have affected the vibration response. For damage mechanisms like shearing and bending, for which the building is forced to follow the vibration motion of the ground surface, the deviation in the buildings fundamental natural frequencies should be of minor importance. However, since amplification occurs when the dominant excitation frequency approaches the buildings natural frequencies, the deviation in the fundamental natural frequencies may affect the building amplification. Hence, for more

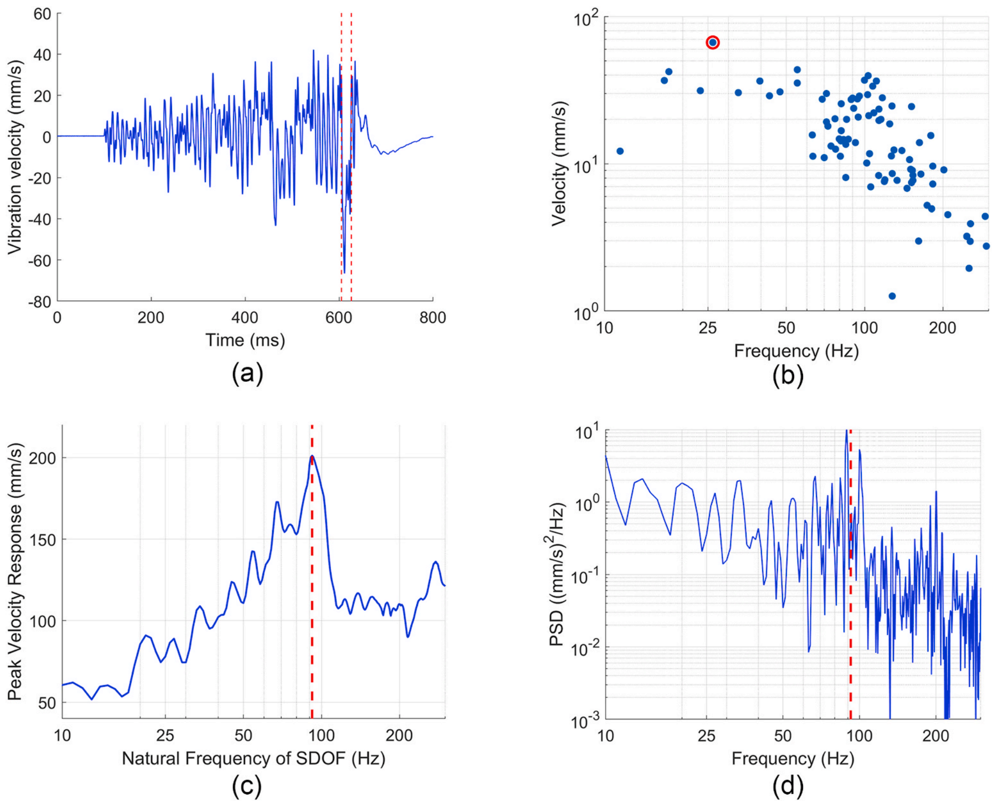

Fig. 7. Measured vibration velocity from the third blast round on ground (Mp3). a) Time series. The instantaneous frequency was computed to 20 Hz around the highest peak, i.e. between the two red dotted lines. b) PPV vs zero-crossing frequency determined by the measurement instrument. c) Response spectra d) Power Spectral Density. The dominant frequency determined by the different methods are marked with a red circle in (b) and red dotted lines in (c) and (d). (For interpretation of the references to colour in this figure legend, the reader is referred to the Web version of this article.)

Table 10
Frequency in vertical direction (Hz), determined by the instrument (zero-crossing around the highest peak) and characteristic frequency determined from the power spectrum of the entire time series.

Blast round	LECA (pos 7-vert)		Concrete (pos 7-vert)		Ground (Mp3)	
	Instr	f_{char}	Instr	f_{char}	Instr	f_{char}
1	80	93	86	91	100	116
2	50	59	46	52	65	100
3	52	65	51	59	26	92
4	22	44	47	52	41	97
5	22	46	28	49	21	45

Table 11
Fundamental frequencies for the test buildings (Hz). Results from hammer excitation in parenthesis.

Building	X-normal to long side	Y-normal to short side
LECA	11 (11)	32 (36)
Concrete	14 (14)	26 (28)

common building structures amplification may be a more dominant damage mechanism. This needs to be further considered before using the findings from this experiment to adjust the vibration limit values in standards. It is particularly important since measurements in accordance with most national standards are specified to be carried out at or close to foundation, which means that possible amplification will not be captured.

13. Conclusion

This field blast experiment has contributed to increased understanding of vibration generation and propagation from bench blasting in rock, response of buildings to ground vibration and vibration damage mechanisms for concrete and light weight aggregate masonry buildings. An extensive set of high-quality synchronized vibration- and strain measurement data are made available from a series of well controlled, well documented rock blast rounds.

The blast tests produced vibration values above PPV = 260 mm/s and strain levels above >1750 μstrain, which is well above the current guideline limit values for vibrations used in most countries and above critical strain levels reported in earlier studies. Despite this, no visible damage was found in any of the two buildings.

Strain calculated from shear wave velocity in ground, measured PPV and frequencies on the buildings, agrees fairly well with strain measured on the homogeneous walls.

Dominant frequency of the vibrations was determined to be between 50 Hz and 130 Hz, which is lower than what can be assumed from the ground condition factors used in the present Norwegian Standard. The dominant frequency was determined by different methods and the results showed a considerable deviation, with a distinct difference between methods which determine the frequency in a short time interval around the highest peak and methods which are using the entire vibration time series. Further, methods which determines the frequency in short time intervals show a large spread in the frequency between the different vibration cycles. This points to the difficulty of using frequency dependent vibration limit values.

The results of the test indicate that today's vibration guideline limit values include a large safety margin for buildings on rock, when considering damages to outer walls, which this study was designed to investigate.

CRediT authorship contribution statement

K.M. Norén-Cosgriff: Conceptualization, Methodology, Software, Validation, Formal analysis, Investigation, Data curation, Writing - original draft, Writing - review & editing, Visualization, Project administration, Funding acquisition. **N. Ramstad:** Conceptualization, Methodology, Investigation, Writing - review & editing, Project

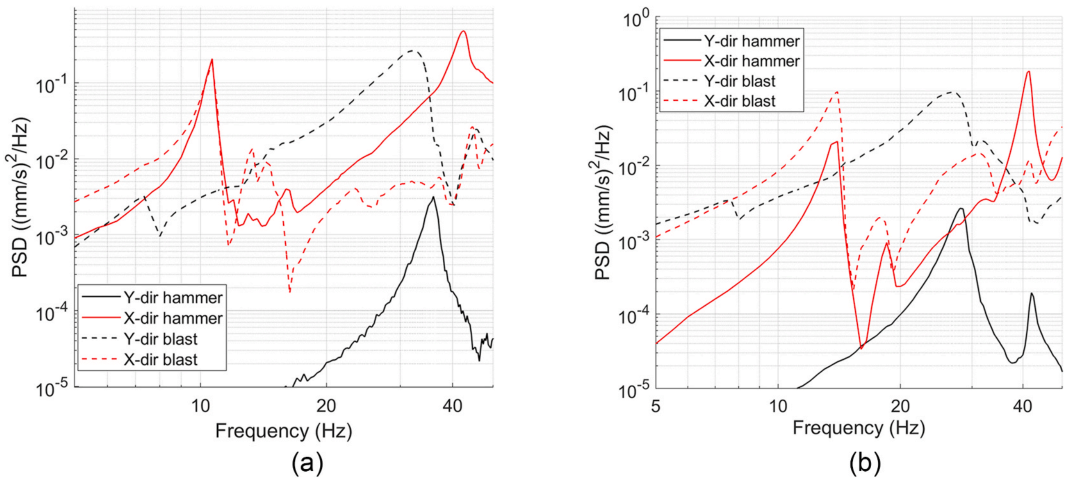

Fig. 8. Building response Power Spectral Density (PSD) from hammer excitation (solid lines) and from blast round 2 (dotted lines). Left, LECA building. Right, concrete building. See Fig. 3(a) for coordinate system.

Table 12
Amplification factors from ground to building (Mp3) and from position close to foundation to building (Pos 7).

Blast round	LECA			Concrete		
	Found	Ground	Pos and dir	Found	Ground	Pos and dir
1	1.5	2.2	4-Z	1.4	1.4	1-X
2	1.2	2.2	6-Z	1.0	1.8	6-Z
3	1.4	1.3	6-Y	1.0	0.9	3-X
4	1.2	1.7	6-Y	1.3	1.4	6-Y
5	–		–	–		–

administration, Funding acquisition. **A. Neby:** Conceptualization, Methodology, Investigation, Resources, Writing - review & editing. **C. Madshus:** Conceptualization, Methodology, Writing - review & editing, Supervision.

Declaration of competing interest

The authors declare that they have no known competing financial interests or personal relationships that could have appeared to influence the work reported in this paper.

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References

[1] Langefors U, Kihlström B, Westerberg H. Ground vibrations in blasting. Water Power February 1958:335–8. 390-395, 421-424.
[2] Edwards AT, Northwood TD. Experimental studies of the effects of blasting on structures. Engineer Sept. 30. 1960;210:538–46.
[3] Nicholls HR, Johnson CF, Duvall WI. Blasting vibrations and their effects on structures. Bulletin, US Department of Interior, Office of surface mining Reclamation and Enforcement656; 1970.
[4] Duvall WI, Fogelson DE. Review of criteria for estimating damage to residences from blasting vibrations. In: Report of investigations 5968. US Department of the Interior, Office of surface mining Reclamation and Enforcement; 1962.
[5] Siskind DE, Stagg MS, Kopp JW, Dowding CH. Structure response and damage produced by ground vibration from surface mine blasting. In: Report of investigations 8507. US Department of Interior, Office of surface mining Reclamation and Enforcement; 1983.
[6] Holmberg R, Lundborg N, Rundqvist G. Soil vibrations and damage criteria. https://doi.org/10.4224/20358507.
[7] Bogdanoff I, Larsson T, Nilsson R. Lättbetonghus utsatt för vibrationer från sprängning. Byggforskningen 1975;R42 (In Swedish).
[8] Bergling JO, Eklund K, Sjöberg C. Betong – lättbetonghus utsatt för vibrationer från sprängning. Byggforskningen 1977;R32 (In Swedish).
[9] Singh PK, Roy MP. Damage to surface structures due to blast vibration. Int J Rock Mech Min Sci 2010;47:949–61.
[10] Dayed-Ahmed EY, Naji KK. Status Quo and critical review of PPV safe limits for subsurface construction blasting close to low-rise buildings. In: Proceedings of the fifth international conference on structural engineering, mechanics and computation (SEMC 2013), cape town, South Africa; 2-4 September 2013. p. 93–8.
[11] Gad EF, Wilson JL, Moore AJ, Richards AB. Effects of mine blasting on residential structures. J. Perform. Construct. Facil. ASCE 2005;19(3):222–8.
[12] Bayraktar A, Türker T, Altunişik AC, Şevim B. Evaluation of blast effects on reinforced concrete buildings considering Operational Modal Analysis results. Soil Dynam Earthq Eng 2010;30:310–9.
[13] NS 8141. Vibration and shock - Measurement of vibration velocity and calculation of guideline limit values in order to avoid damage on construction. Standard Norge; 2001 (In Norwegian).
[14] Bs 7385-2. Evaluation and measurement for vibration in buildings – Part 2: guide to damage levels from groundborne vibration. 1993.
[15] Din 4150-3:2016-12, Erschütterungen im Bauwesen – teil 3: einwirkungen auf bauliche Anlagen ("Vibrations in buildings – Part 3: effects on structures").
[16] U.S. Code of federal regulations (CFR) 30 §715.19, §816.67 and §817.67.
[17] Hao H, Wu C. Numerical study of characteristics of underground blast induced surface ground motion and their effect on above-ground structures. Part II. Effects on structural responses. Soil Dynam Earthq Eng 2005;25:39–53.
[18] Rundqvist G. Naturlig sprickbildning i nya småhus – hjälpmedel för besiktning efter sprängning. SBUF Projekt nr 2139. 1994 (In Swedish).
[19] Dowding CH, Aimone-Martin CT. Micro-meter crack response to rock blast vibrations, wind gusts & weather effects. Proc. Geo-Denver 2007 February;2007: 18–21. Denver, Colorado, United States.
[20] Sintef Byggforsk. Bergmekaniske egenskaper - Testing av borekjerner boret normalt på/parallel med foliasjon fra testsprengningsområde Solør vgs. avd Våler Steinbrudd, Spulsåsen. Report no 18141BM. 2019.
[21] https://leca.no/teknisk-informasjon/leca-teknisk-handbok/.
[22] http://www.micronoptics.com/wp-content/uploads/2018/01/os3510-1.pdf.
[23] SS 025210. Vibration and shock - blast induced airborne shock waves - guidance levels for buildings. SIS; 1996 (In Swedish).
[24] Cartwright DE, Longuet-Higgins MS. The statistical distribution of the maxima of a random function. In: Proceedings of the royal society of london, series A. vol. 237; 1956.
[25] Vuolio R. Blast vibration : threshold values and vibration control. In: Acta polytechnica scandinavia, civil engineering and building construction series No. vol. 95; 1990. Helsinki.
[26] Norén-Cosgriff K, Rothschild S. Sammenligning av ny og gammel NS8141 (in Norwegian), http://nff.no/wp-content/uploads/2016/04/10b-Noren-Cosgriff-Sammenligning-ny-gammel-NS-8141.pdf.

[27] Ambraseys NN, Hendron AJ. Dynamic behavior of rock masses. In: Stagg K, Zienkiewicz O, editors. Rock mechanics in engineering practice. London: John Wiley & Sons; 1968. p. 203–36.
[28] Bendat JS, Piersol AG. Random data: analysis and measurement procedures. fourth ed. John Wiley & Sons, Inc; 2010.
[29] Westine PS, Dodge FT, Baker WE. Similarity methods in engineering dynamics – theory and practice in scale modelling - revised edition. Elsevir; 1991.