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VIBRATION MITIGATION FOR METRO LINE ON SOFT CLAY

Karin Norén-Cosgriff Norwegian Geotechnical Institute, Oslo, Norway
Christian Madshus Norwegian Geotechnical Institute, Oslo, Norway
Arild Brekke Brekke & Strand Akustikk AS, Oslo, Norway

1 INTRODUCTION

Trains and trams generate vibrations as they pass over local variations along their track. These vibrations propagate from the track through sleepers, ballast and foundation to the ground and are subsequently transmitted to the structures of nearby buildings. The low frequency part of the vibration can be felt by occupants as whole body vibration due to shaking of the buildings. The more high frequency vibrations can be re-radiated from the building surfaces as felt as audible structure- or ground borne noise.

As part of the planning work for upgrading the existing metro line Kolsåsbanen outside Oslo to modern standards, calculations of expected low frequency vibrations and structure borne noise were performed. The calculations showed that low frequency vibration velocity levels higher than the acceptance limit could be expected for parts of the metro section under consideration. A mitigation solution based on lime cement column soil stabilization beneath the track was suggested. However, experience gained from other sites where lime cement column stabilization has been introduced show that this low frequency vibration countermeasure may give rise to increased structure borne noise levels. Since structure borne noise is of as much concern as low frequency vibrations for nearby buildings, the project wanted to combine measures against low frequency vibrations, i.e. lime cement column stabilization, with measures against structure borne noise based on sub-ballast mats. Since there is limited experience with such combinations it was decided to perform a dynamic FE-analysis to verify the effect of the solution. Another uncertain factor which needed to be verified is how lime cement column stabilization performs when the soil depth from track foundation to bedrock is short, e.g. typically 10 m, as is the case for parts of the Kolsåsbanen.

2 VIBRATION ACCEPTACE STANDARDS IN NORWAY

Norwegian building vibration acceptance criteria from land based transport are formulated in a national standard NS 8176[1]. The standard gives limits of acceptable vibration velocity in four classes where Class C corresponds to the recommended limit value for vibration in new residential buildings and in connection with the planning and building of new or upgraded transport infrastructure. The recommended limit for Class C is $v_{w,95}$ = 0.3 mm/s where $v_{w,95}$ is the statistical maximum weighted vibration value, i.e. the vibration value that can be expected to exceed by no more than 5% of the train passages. Frequency weighting is performed according to ISO 8041[2]. The vibration acceptance criteria in the Norwegian standard is based on an extensive socio-vibrational survey, where about 1500 people living along roads and train-, tram- and metro limes in Norway were interviewed about their experience of the vibration situation in their homes[3,4]. The corresponding vibration impact in the apartments was determined partly by measurements and partly from an empirical prediction model. Results were treated statistically through an ordinal logit and logistic regression model to establish exposure-effect relationships. The result is shown in Figure 1. The Class C criterion of $v_{w,95}$ = 0.3 mm/s is set where about 15% of people are highly and somewhat annoyed by the vibration.

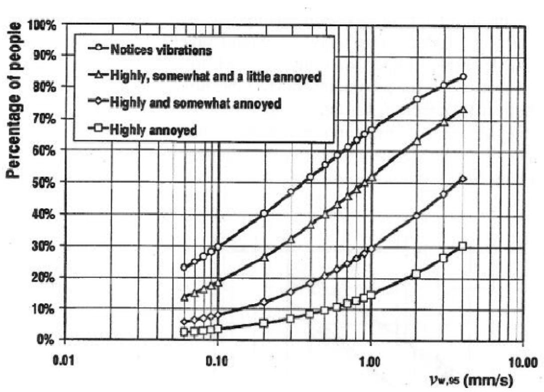

Figure 1: Results from the Norwegian socio-vibrational survey on human response to vibration in homes. Estimated cumulative percentage of people reporting different degrees of annoyance by the strength of vibration $v_{w,95}$.

According to Norwegian standard NS 8175[5], the acceptance criterion for ground (structure) borne noise in sleeping- and living rooms is $L_{Amax,Fast}$=32dB.

3 NUMERICAL SIMULATION MODEL

The analysis was performed using the FE-program Comsol Multiphysics[6]. Characteristics of materials have been derived from geotechnical site investigations performed by the geotechnical consultant of the project and laboratory tests of typical filling materials performed in an earlier project by NGI for the Norwegian Rail Administration[7].

The FE-model is 2-dimensional, describing a track cross-section representing a typical 1 m thick slice perpendicular to the track. In a 2-D model the vibrations will not decline with distance as much as in reality. However, since the aim of the study was to compare relative effects of different vibration mitigation measures and not to determine absolute vibration values, the use of a 2-D model was found to be adequate. The extension of the model is 158 m with the tracks centered in the middle of the model. The calculations have been performed in the frequency domain. The element type is quadratic (second order) Lagrange elements with a maximum element size of 0.4 m for frequencies up to 20 Hz and 0.15 m for frequencies from 20 to 63 Hz. With typical shear wave velocities in the clay between 80-120 m/s this is considered to be appropriate. The models have transmitting boundaries (PML – perfectly matched layers) in order to allow the vibration energy to spread out of the model and prevent it from reflecting back from the boundaries.

3.1 Geometry

The rail, foundation mitigation solution as specified in Table 1 has been modeled as the basic case. The constituents of the model are listed in the table from above top of rail and downwards:

Table 1: Constituents in model for the basic case

No	Constituent	Thickness [m]	Part	Lime Cement Columns
1	Unsprung mass of wheel and shaft	-	Train	
2	Rail	-	Track and substructure	
3	Sleeper	-		
4	Ballast	0.5		
5	Sub-ballast mat	0.085		
6	Crushed rock	0.3		
7	XPS	0.06		
8	Crushed rock	0.1		
9	Quarry dust	0.3	Ground	**X**
10	Dry crust	1.0		**X**
11	Silty clay	2.5		**X**
12	Soft quick clay	6.0		**X**
13	Soft quick clay	1.0/5.0/8.0		
14	Bedrock	-		

The lime cement columns have a typical shear strength of 625 kPa, a diameter of 600 mm and centre to centre distance of 1.5 m. Calculations have been performed for the following models:

Table 2: Models made subject to analyses

Model no	Model name	Model coverage
1	Basic model	Basic model as described in Table 1 and shown in Figure 2. The lime cement columns have a distance between rows of 1.0 m in the direction along the track.
2	Reference model	Like the basis model but without lime cement columns.
3	Columns, no mats	Like the basis model but without sub-ballast mats.
4	Alternating columns and mats	Like the basis model but with varying configuration and length of the lime cement columns (alternating 6 m and 10 m) as shown in Figure 3. The long lime cement columns do overlap in the direction along the track.
5	Alternating columns, no mats	Like Model 4 but without sub-ballast mats.

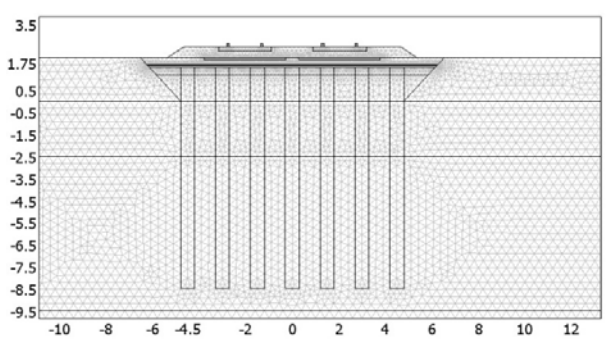

Figure 2: Finite element outline applied for the Basic model (Model 1), including lime cement columns and sub-ballast mats.

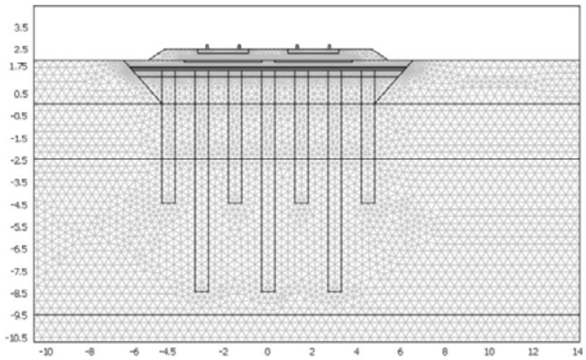

Figure 3: Finite element outline for Model 4, including lime cement columns of alternating length and sub-ballast mats.

3.2 Material parameters

Geotechnical sub-surface surveys including soil sampling has been performed in two positions along the section in question. Material parameters to the model have been derived from survey results, laboratory tests and previous experience [7,8,9]. Material parameters used in the models are shown in Table 3.

Table 3: Material parameters

Material	E–mod [2)] [MPa]	ν	ρ [kg/m^3]	η =1/Q
Rail	205E3	0.28	49 (kg/m)	0.02
Concrete sleeper	25E3	0.33	2300	0.02
Ballast	150-180 [1)]	0.25	1500	0.3 [2)]
Subballast mat	2.13 [3)]	0.1	229	0.2
Gravel/ quarry dust	110-190 [1)]	0.25	1450	0.3 [2)]
XPS	20	0.1	38	0.1
Dry crust	66	0.3	1900	0.08
Silty clay	38-66 [1)]	0.4991	1900	0.06
Quick clay	38-82 [1)]	0.4991	1900	0.06
Lime cement column in clay	1000	0.33	2000	0.1

1) Varies with depth from surface, related to shear strength.
2) Based on experience from laboratory tests performed for Norwegian Rail Administration.
3) Corresponds to k_{dyn} = 0.025 N/mm^3 and a subballast mat thickness of 85 mm.

3.3 Added mass from wheel shaft and load from train

The unsprung mass of the wheel shaft has been introduced in the models as increased rail density for one of the two tracks. As mentioned before the 2-D model does assume a 1 m thick cross section perpendicular to the track direction. Therefore this segment has been given an additional mass of two wheel shafts and this mass has been distributed over the two rails of the loaded track.

The total weight of the whole train is only indirectly included in the calculation model. Granular materials like ballast, crushed rock and quarry dust, as well as the sub-ballast mats have a stress-dependent non-linear behavior. The dynamic parameters of these materials used in the model are chosen in a way to account for the effect of added vertical (and horizontal) stress due to the weight of the train which is considered quasi-static during the duration of is pass-by.

3.4 Dynamic force excitation

The model has been excited by a vertical dynamic unity force over the entire frequency range of interest. The force has been applied synchronously on both right and left rail of one of the tracks at the mid frequencies of each 1/3-octave frequency band from 1.0 to 63 Hz.

4 RESULTS OF NUMERICAL SIMULATION

The calculation results are shown as amplitudes of normalized particle velocity of vertical vibration. In order to reduce the influences of local phenomena, which can be dominant in the dry crust especially for the higher frequencies, we have from the FEM output determined the average value over the depth of the dry crust and also the average value over a 6 m long section along the surface with its closest point at 10 m distance from centre line between the two tracks. This has been done in order to give a realistic picture of the vibrations a building foundation close to the line will respond to.

The results for all frequencies are shown in Figures 4 through 6. The plots to the left compare the various mitigation measures shown as the ratio between calculated vibration velocity for the model including the mitigation measure in question and calculated vibration velocity for the reference case (Model 2). A ratio of 1.0 corresponds to no effect of the mitigation measure. A ratio higher than 1.0 corresponds to increased vibration levels as result of the mitigation measure, i.e. a “negative” effect. The plots to the right present calculated vertical normalized vibration velocity at the various frequencies, and are determined when the models are excited by a unity force over the entire analyzed frequency range. The values are not comparable to vibration values seen during a real train pass by since all frequencies have been excited with an equal unity force in the FE-model. What the figures show is at which frequencies the ground response (including mitigation measures) to excitation from trains is highest. The vibration velocity has been further normalized by dividing the results for all models with the highest vibration velocity for the reference model (Model 2).

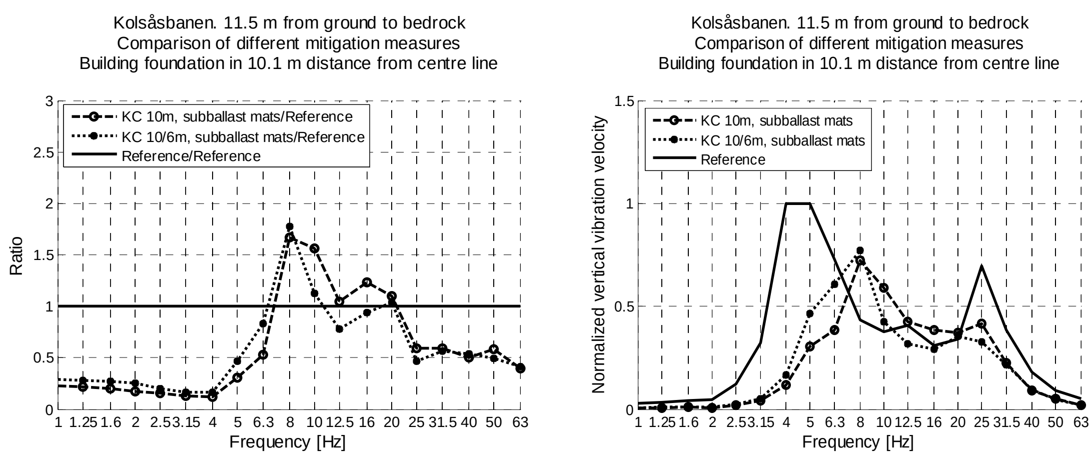

Figure 4: Case with 11.5 meter soil depth from track foundation to bedrock. Comparison of lime cement column stabilization (denoted KC in the figure legends) with the reference solution without mitigation measures. Dashed lines show Model 1, dotted lines Model 4 and solid lines Model 2 (reference model).

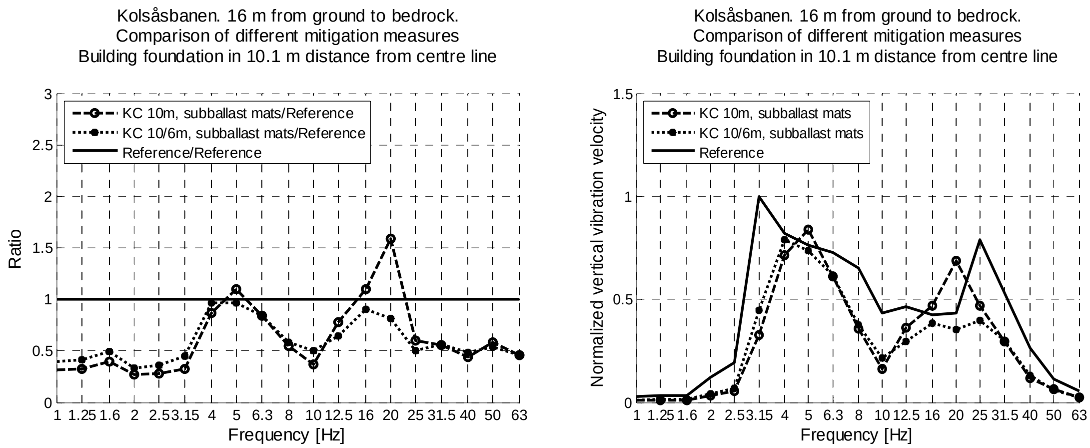

Figure 5: Case with 16 meter soil depth from track foundation to bedrock. Comparison of lime cement column stabilization (denoted KC in the figure legends) with the reference case without mitigation measures. Dashed lines show Model 1, dotted lines Model 4 and solid lines Model 2.

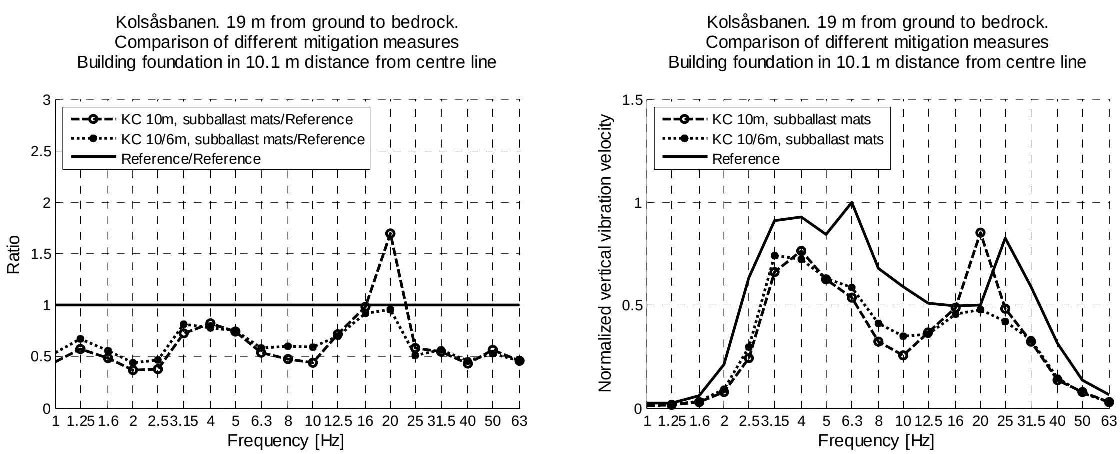

Figure 6: Case with 19 meter soil depth from track foundation to bedrock. Comparison of lime cement column stabilization (denoted KC in the figure legends) with the reference case without mitigation measures. Dashed lines show Model 1, dotted lines Model 4 and solid lines Model 2.

5 COMMENTS TO RESULTS AND CONCLUSIONS

Vibration measurements performed along Kolsåsbanen before the upgrading work started shows that most vibration velocity spectra has a first peak in the frequency range between 8 and 12.5 Hz, and distinctly drops off towards lower frequency, as can be seen from Figure 7.

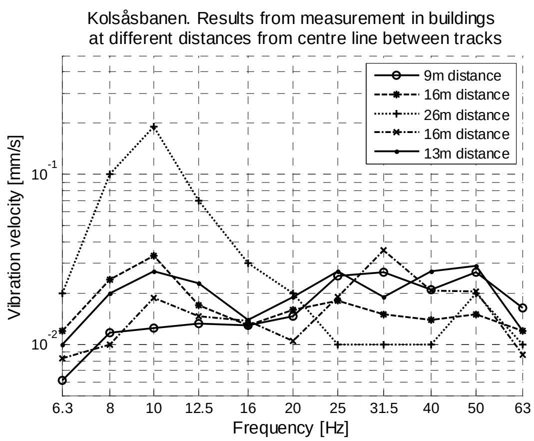

Figure 7: Vertical vibration particle velocity measured in nearby buildings along Kolsåsbanen at low speed before upgrading. Measurements are taken at various locations and different distances from the track, shoving typical frequency content and demonstrating large variability due to local ground condition and building foundation.

The performed FEM calculations show that lime cement column stabilization shifts the first resonance of the ground to a higher frequency. For the larger soil depth from track foundation to bedrock (16 m and 19 m) this is not of concern since the resonance frequency will still be below 5 Hz and hence not in the region for major vibration energy from the passing trains, as shown in Figure 7. For the situation where the depth from track foundation to bedrock is short (11.5 m) the calculations do however show that the first resonance is shifted from about 4-5 Hz to about 8 Hz and hence into the frequency region of excitation from trains. However, the calculations do also show that by varying the configuration and length of the lime cement columns the frequency range where the vibrations are amplified can be reduced.

Further, the FEM calculations show that sub-ballast mats will reduce the vibrations at higher frequencies which can cause structure borne noise. Note however that use of sub-ballast mats will introduce a new slight increase in the vibration spectra caused by the resonance of the composed system: mass of track and ballast on the spring that the sub-ballast mats constitute. This effect can be seen between 12.5 and 20 Hz in Figure 8. Above 20 Hz the mitigation effect of the sub-ballast mats is obvious. The calculations do also show that by varying configuration and length of the lime cement columns this peak can be reduced, see Figures 4 through 6.

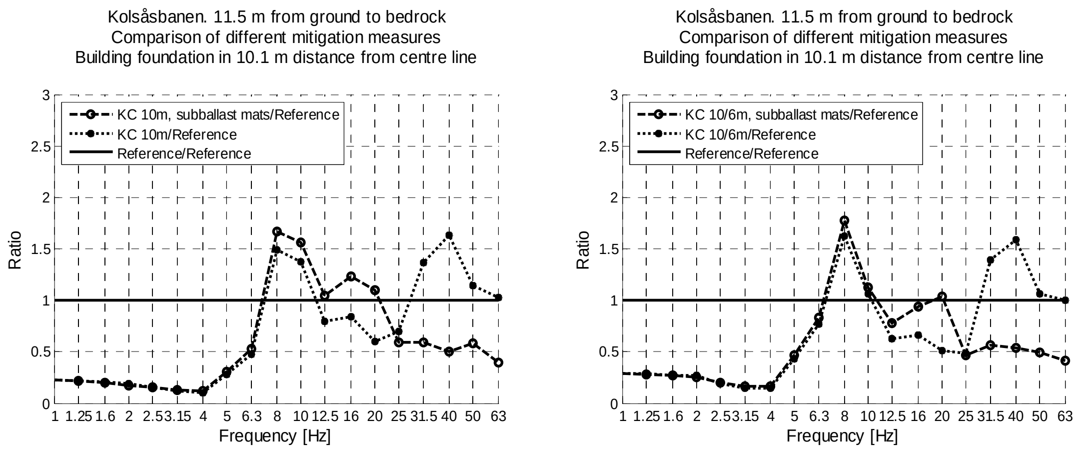

Figure 8: Comparison of solution with lime cement columns and sub-ballast mats (dashed line), with lime cement columns without sub-ballast mats (dotted line) and the reference case (Model 2). Figure to the left show situation with 10 m long lime cement columns and the figure to the right shows the situation with alternating length and configuration of columns.

6 ACKNOWLEDGEMENTS

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7 REFERENCES

1. NS 8176.E:2001 Vibration and shock Measurement of vibration in buildings from landbased transport and guidance to evaluation of its effects on human beings.
2. ISO 8041:2005 Human response to vibration -- Measuring instrumentation.
3. Turunen-Rise, I.H.; Brekke, A.; Hårvik, L.; Madshus, C.; Klæboe, R. (2003). Vibration in dwellings from road and rail traffic. Part I. A new Norwegian measurement standard and classification system. Applied Acoustics, Vol. 64, 2003, No. 1, pp. 71-87
4. Klæboe, R.; Turunen-Rise, IH.; Hårvik, L.; Madshus, C. (2003). Vibration in dwellings from road and rail traffic. Part II: Exposure-effect relationships based on ordinal logit and logistic regression models. Applied Acoustics, Vol. 64, 2003, No. 1, pp. 89-109.
5. NS 8175:2008 Acoustic conditions in buildings. Sound classification of various types of buildings (in Norwegian)
6. Comsol Multiphysics: http://www.comsol.com
7. Madshus, C., Rothschild, K., Cleave, R., Brekke, A. (2008). Full scale tests on mitigation of ground borne noise from railway tunnels in rock. Workshop, Noise and Vibration on High-Speed Railways, Porto, Portugal, 2-3 october
8. Andersen,K.H., A. Kleven and D. Heien (1988). Cyclic Soil data for Design of Gravity Structures. ASCE Journal of Geotechnical Engineering. Vol 114, No.5, May, 1988
9. Larsson,R. and M.Mulabdic' (1991). Shear moduly in Scandinavian clays. Swedish Geotechnical Institute Report no 40, Lindköping