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Impact of sound and vibration of the North–South high-speed railway connection through the city of Antwerp Belgium

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Abstract

In the European High-Speed Train Network the infrastructure of the North–South connection in Antwerp needs significant modifications. For the section between Berchem and Antwerp Central Station the existing track on the high level embankment will be incorporated into concrete structures providing a three level track access to the station. For the section between Antwerp Central Station and Dam two drilled tunnels are planned providing the station with pass-through facilities instead of being an "end" station as at present. The paper focuses on the methods of practical research and the resulting measures regarding the impact of sound and vibration on the environment.

An essential part of this study is the impact of the planned construction of a double railway tunnel underneath the city of Antwerp. At certain locations, the distance between the foundations of the houses and the top of the tunnel is only 4 m. The study considers the projected vibration levels on the rail, the tunnel invert, building foundations and upper floors of the buildings. Also the ground-borne noise is evaluated. The study identifies the measures necessary at the rail mounting level. As a result, a floating slab has been proposed and the effects on the environment are estimated.

1. General project information

Following the agreements made at a European level, Belgium is currently modifying its railway infrastructure for the implementation of the high-speed train (HST) network between Belgium, France, Germany and the Netherlands. The existing railway line between Brussels and Amsterdam passes through Antwerp and for several reasons it was decided that the HST line should call at Antwerp Central Station rather than at Berchem. However, as there is currently no

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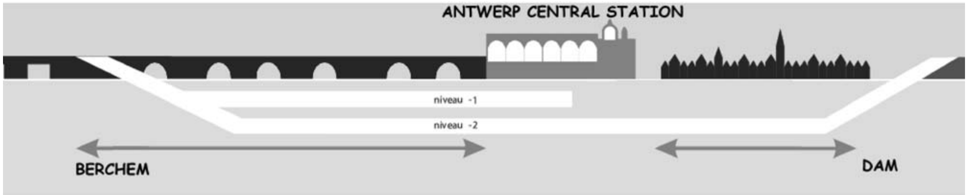

Fig. 1. Basic cross-section of the North–South railway project through Antwerp Belgium.

line to take trains further through the city, the HSTs would have to travel back to Berchem and take the existing line around the city. The alternative is to construct a new tunnel between the Central Station and the future Dam Station in the north of Antwerp. The latter option was chosen (Fig. 1).

This new North–South connection through Antwerp can be subdivided into the connection between Berchem (South) and Antwerp Central and the connection between Antwerp Central and Dam (North).

2. The connection Berchem–Antwerp Central

The track is at present elevated above ground with respect to the vicinity. It crosses many streets and the structures have a historic value and are protected. In the future this single level has to be further subdivided into three levels near the central station, mainly because the rail level has to be about 15 m below ground level so that it can connect with the future tunnel. Furthermore, the present structures have to be preserved.

At this location, the dynamic problem is two-fold. Firstly, environmental vibration nuisance due to trains has to be minimized and secondly, future vibration levels of the older structures, which are to be integrated into the future structures, have to be such that no vibration damage would occur.

To be able to transfer the incoming trains on to the three levels of tracks, many crossings and points are necessary. These are grouped in two main interchange complexes before reaching the Central Station. These areas are regarded as the most critical for vibrations due to points and crossings. Therefore these sections have to be studied more in detail.

At first an estimate was made of the resulting dynamic forces based on actual measurements, on a different location at Lot–Beersel. These measurements establish the transfer functions between point of excitation at rail level and a number of points in close to the track (Fig. 2). Also the vibration levels were recorded during actual train passages.

By analysing and combining the results of these transfer functions and vibration level recordings, the dynamic force at rail level that would result in the same vibration levels can be deduced. Assuming that the shock has a half-sine wave shape, the dynamic force amounts to 110 kN lasting 6.4 ms at the rail crossings.

The next step was to measure current vibration levels on the protected buildings and monuments as a result of train passages as points of reference. Following the German standard DIN 4150 part III [1], the maximum allowable vibration levels are structure type dependent

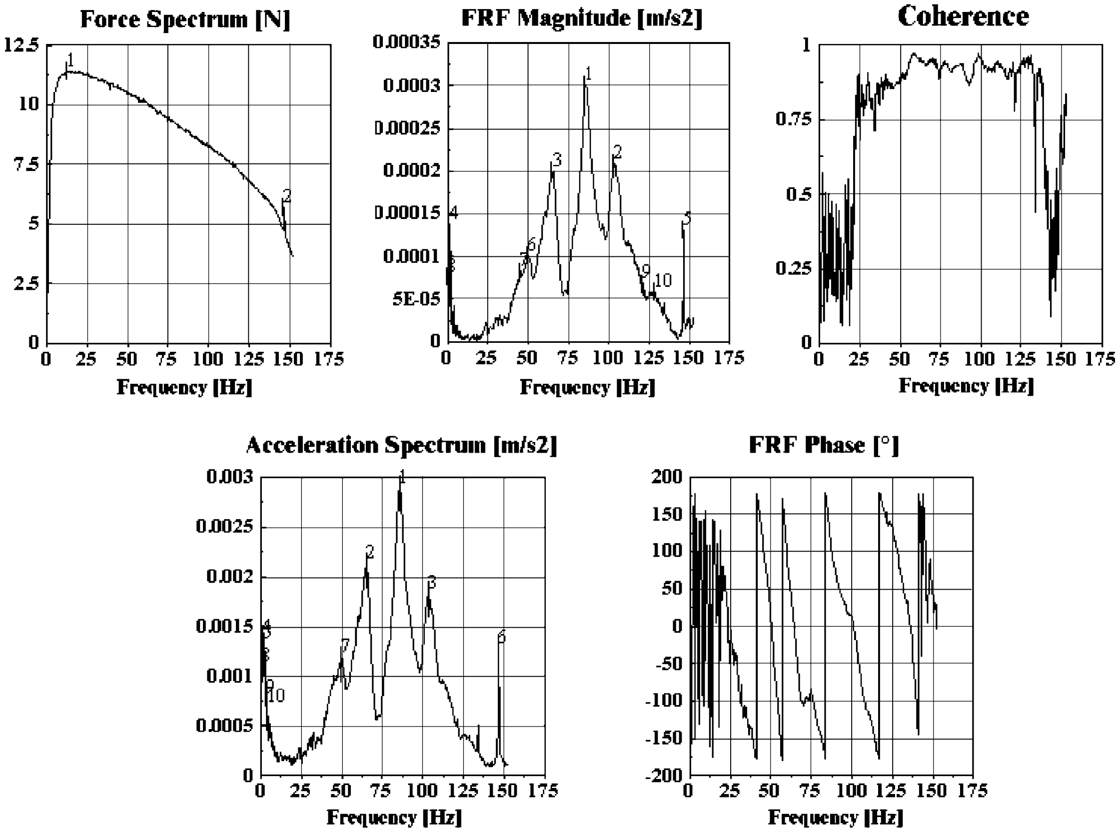

Fig. 2. Frequency response function alongside the track due to shock excitation on the rail.

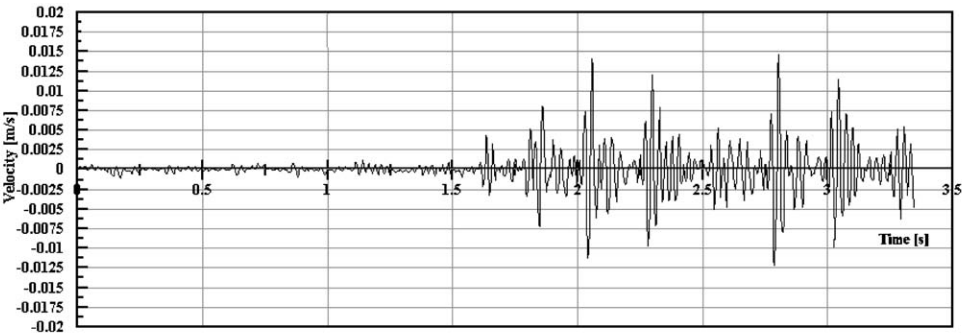

Fig. 3. Vibration velocity (vertical direction) on historical structure during train pass-by.

and frequency dependent. For monuments (historic buildings), the limits range from 3 to $10\,\mathrm{mm\,s^{-1}}$ increasing with the dominant frequency of the vibration. The measurement programme shows that at present, in the most unfavourable situation very close to railway crossings (a few meters), up to $20\,\mathrm{mm\,s^{-1}}$ (Fig. 3) has been recorded and thus exceeds the criteria.

Next, the expected impact of vibrations had to be estimated for the future structures. This was estimated from two-dimensional numerical simulations based on design drawings. Based on the estimated number of expected point impact sources along the railway line, the excitation for the two-dimensional model was re-calculated into a line source of retained calculation value of $45\,\mathrm{kN\,m^{-1}}$ [2].

The first interchange complex is situated at the Mercator and Oostenstraat at the intersection with the Lange Van Ruysbroeckstraat. The second interchange complex is situated at the Van Spangenstraat. A two-dimensional finite element model was constructed based on this design (Fig. 4). The construction at this location already contains three structural levels. Note that only a very small part is shown here as the model boundaries are much wider so as to eliminate reflections at boundaries. A plain strain condition was assumed [3]. For the soil parameters, several references are taken but more in particular, Seed and Idriss [4] determine also the material damping of sands at about 1% and of clays at about 2% for very low strains. The dynamic modulus of Young for building materials is taken to be 1.15 times the static value [5].

In the models the track bed consists of classical ballast on a concrete structure. The calculations concentrated on direct time integration. It was shown that the expected vibration levels on the protected structures were less then $2\,\mathrm{mm\,s^{-1}}$ and complied with the German standard DIN 4150 Part III for damage to historic structures.

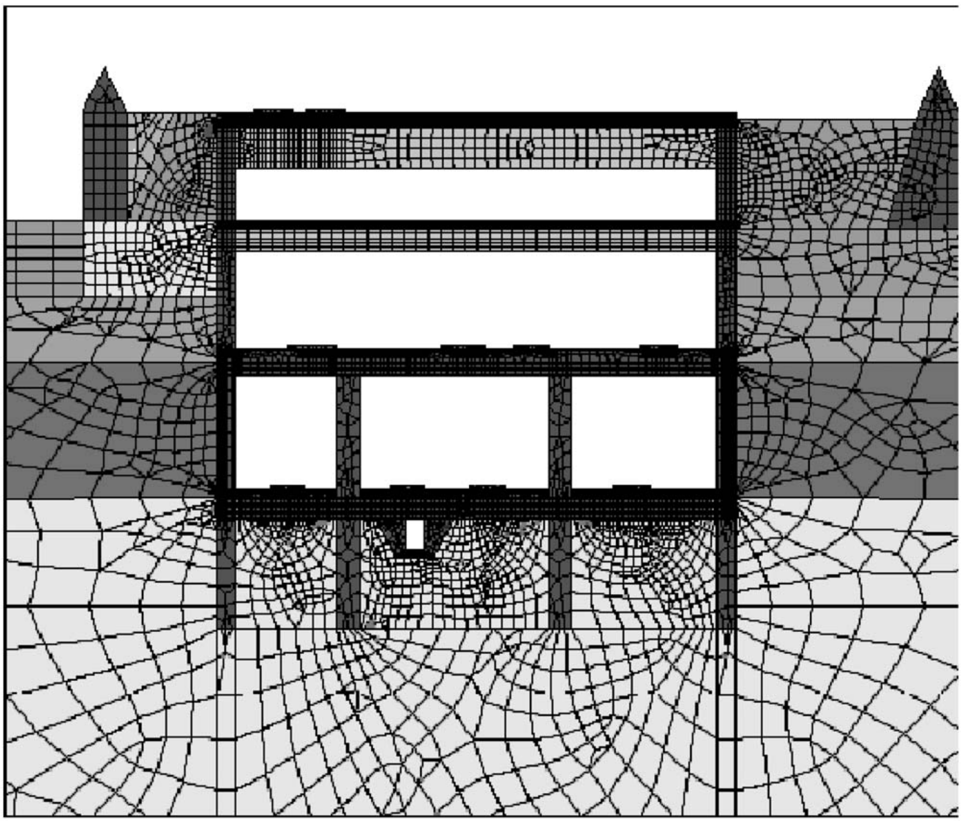

Fig. 4. Structural finite element model at intersection with Lange Van Ruysbroeckstraat.

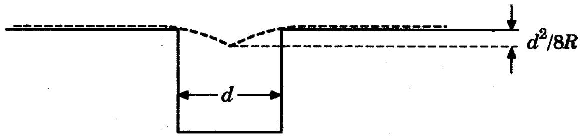

Fig. 5. Relation between vertical axle displacement and gap length.

However, future rail traffic predictions indicate that environmental vibration levels are likely to exceed the levels given in DIN 4150 Part II and therefore additional vibration isolation was needed. Initially this was achieved by using structural modifications. This resulted in expected values for KB_{Ftr} of 0.1 during daytime and 0.05 at night; this was also of course dependent on the point of emission and exposure.

In order to evaluate the necessary supplementary isolation on the railway foundations, an additional one dimensional finite element model was used along the railway line integrating rail and track stiffness and sprung and unsprung mass. Railpad properties were chosen with reference to Vincent et al. [6]. Sleeper properties were based on Grassie [7]. Ballast properties with chosen according to Jones and Block [8].

The variables of the model were tuned so that fundamental eigenfrequencies at the rail level fitted those measured at Lot–Beersel (with and without unsprung mass). An impact excitation was time simulated through a step discontinuity with a 3 cm width (resulting height 0.3 mm depending on wheel diameter) (see Fig. 5). Train passage speed was assumed to be $10\,\mathrm{km\,h^{-1}}$.

Note that the loading of the model is not force based but displacement based. Thus for a certain forced displacement discontinuity more compliance results in lower forces being exerted on the substructure. The variable of interest was thus the elasticity under ballast. Supplementary reduction of forces up to 38.1% is to be expected by using resilient elastomers beneath the ballast bed (Fig. 6).

3. The connection from Antwerp Central to Dam

According to the construction programme, the drilling of the first tunnel will commence during 2003, followed by the second tunnel in the first half of 2004. The internal tunnel construction works are expected to commence during 2005. The tunnel will have an inside diameter of 7.2 m and a wall thickness of 0.4 m (Fig. 7). The rails are continuously welded rails. Due to the interaction of irregularities between the wheel and the rail, forces are exerted on the tunnel lining through the track and track supports. This is dependent of the type of rail support inside the tunnel.

The soil transmits vibrations from the track to the foundations of nearby buildings and structures (Fig. 8). This is dependent on soil consistency and stratification. Within the buildings, vibration levels can be amplified due to structural resonance's coinciding with vibration excitation frequency content. Finally, simulations were made to predict the expected radiated structure-borne noise. Detailed analysis has been performed for the tunnel section at Viséstraat, which is regarded as the most critical location, due to the closeness of the very shallow tunnel to the

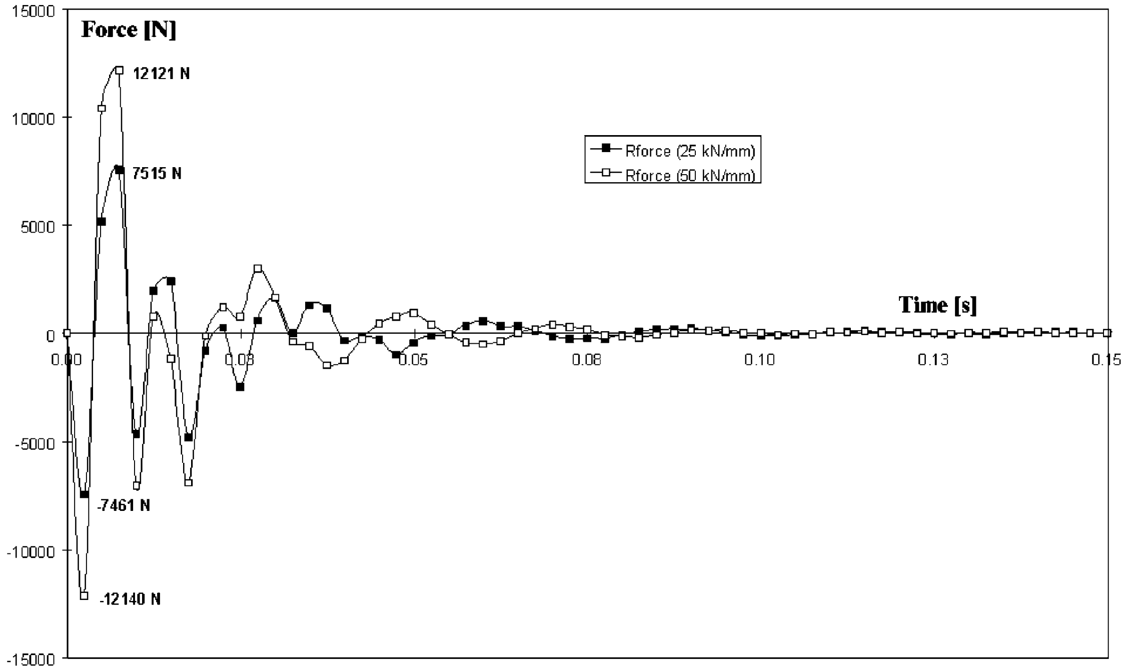

Fig. 6. Reaction forces as function of time due to axle crossing a gap for different resilient elastomers beneath the ballast bed.

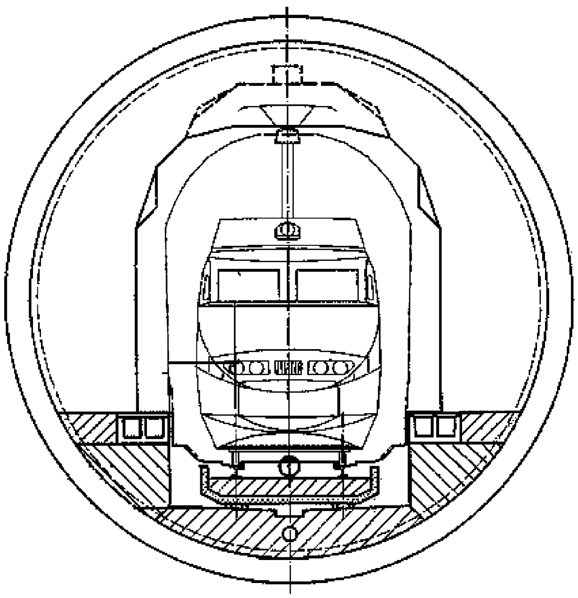

Fig. 7. Principal drawing of cross-section of one drilled tunnel.

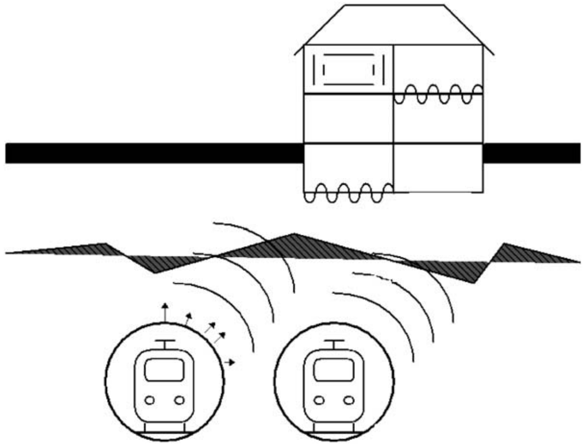

Fig. 8. Principal drawing of different parameters at vibration source, transmission and recipient.

foundations of nearby houses; near this point the line emerges above the ground. Finally, other locations alongside the tunnel were also evaluated.

Initially, a validated estimate had to be made of the irregularity function between the wheel and rail. It was decided to organize a measurement programme at the outskirts of Antwerp because this location had identical geological properties as in the Viséstraat, which was considered to be the most critical part of the tunnel.

Through shock excitation of the rail, transfer functions into the surrounding environment were established. The frequency response function as a function of frequency are shown in Fig. 9.

The dynamic behaviour of the measured frequency response functions were then simulated using an approximate numerical five-degrees-of-freedom (5 d.o.f.) mass–spring–damper model. At the same location a number of train passages were recorded. By introducing the unsprung mass into the model, it is possible to define a relative frequency dependent displacement function between the wheel and the rail, which generates the measured vibrations.

The values of the functions obtained in this way are compared with other references from literature (Fig. 10) as a power spectral density function (PSD). Note that the values obtained are somewhat higher than found in literature. This is primarily explained by the fact that the simultaneous interaction of several wheel–rail interactions are translated into a single excitation function.

For information purposes, for a wavelength of 3.15 cm, an irregularity of 1 μm is obtained, and the irregularity increases to about 100 μm over a wavelength of 1 m. Such values are also recorded on the Dutch railways [9], which also show the possibility of a large spread, of up to ± 15 dB, between values depending on the surface condition of wheel and rail.

The dynamic behaviour of a tunnel in the same geological soil conditions was then examined. In Antwerp, several unused pre-metro tunnels are available. Measurements were arranged at the Herentalsebaan at the Woodstraat crossing (Fig. 11) with a temporary rail-traverse-ballast bedding.

Transfer functions were measured at the tunnel wall and in the local environment. These are shown in Fig. 12. Based on these measurements, the parameters of a 5 d.o.f. simplified model were again constructed to include the dynamic behaviour of the tunnel.

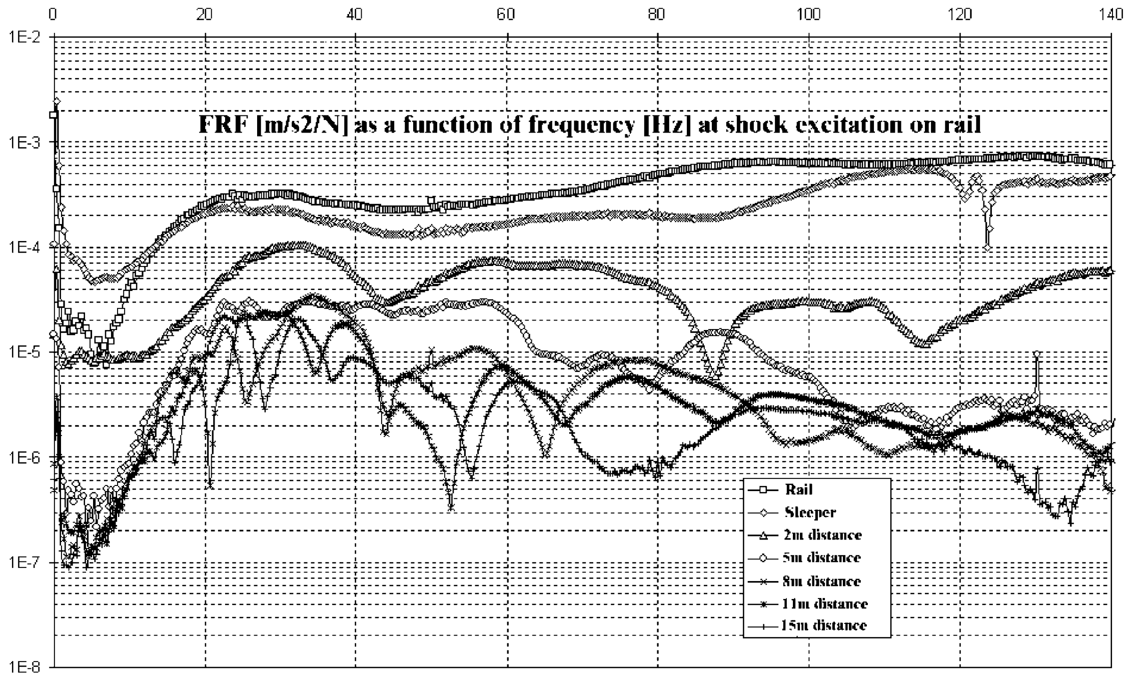

Fig. 9. Frequency response functions as a function of frequency on rail, sleeper and the immediate vicinity due to a shock excitation of the rail.

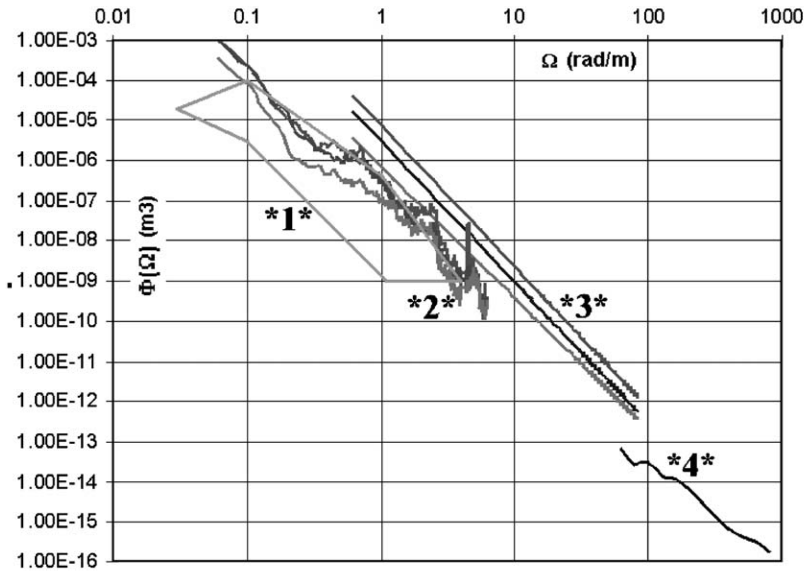

Fig. 10. Power spectral density functions of irregularity at rail level (*1*: region according to Braun VDI, *2*: representative at Belgian Rail (Courtesy NMBS), *3*: at track Colmar Antwerp, *4*: according to P.C. Dings).

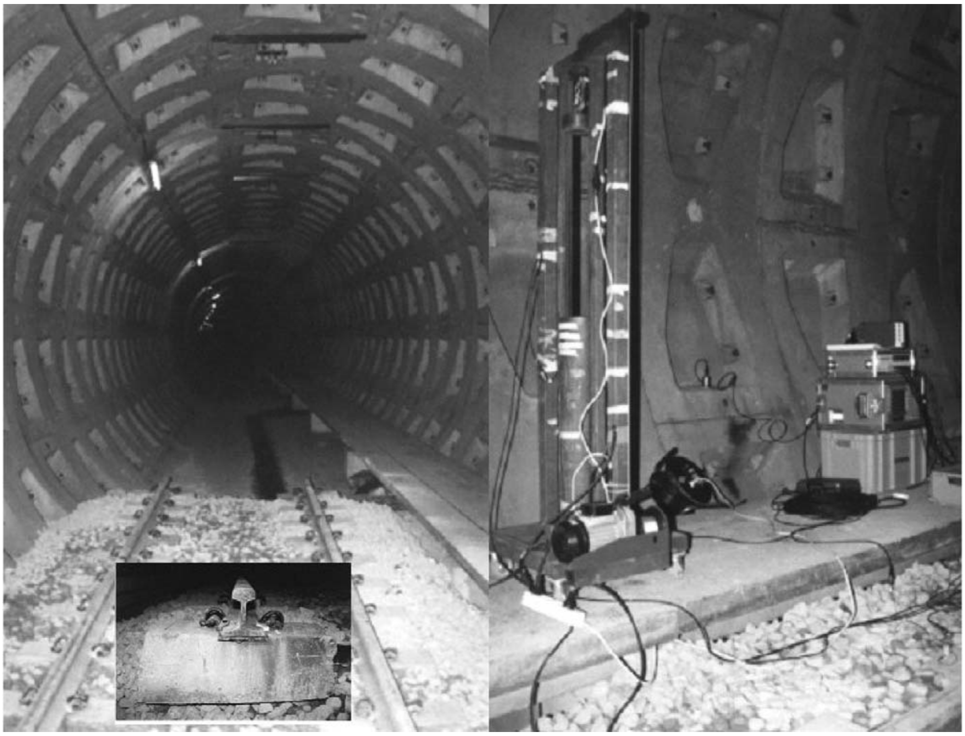

Fig. 11. Measurement set-up at pre-metro tunnel Herentalsebaan (Woodstraat crossing).

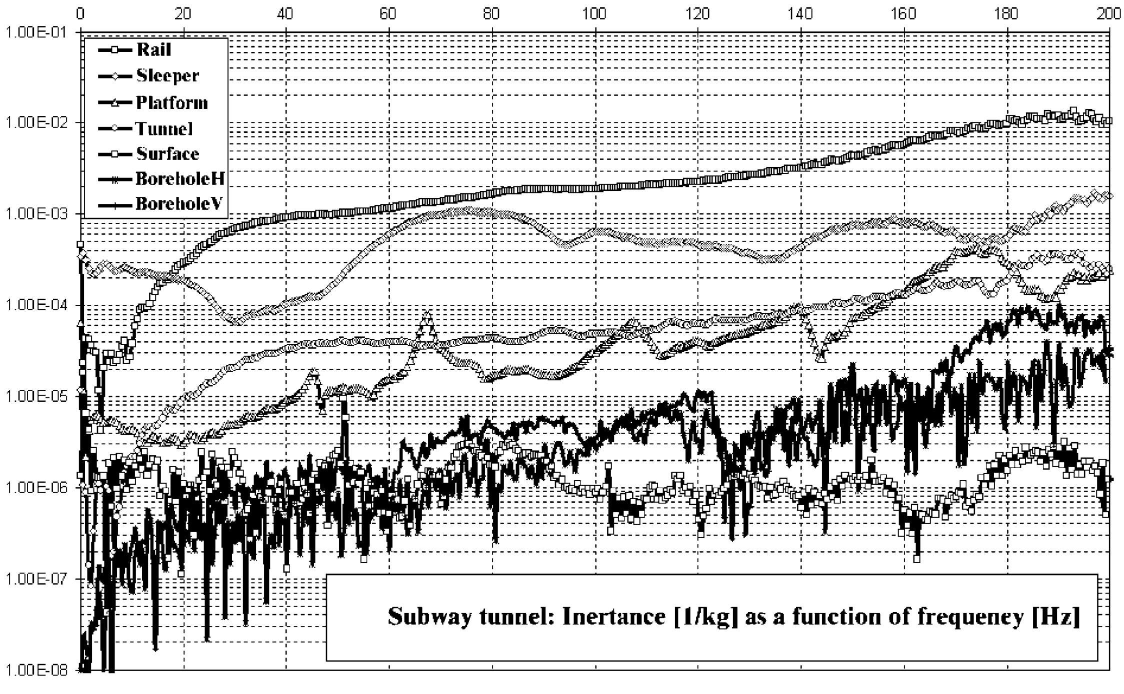

Fig. 12. Transfer functions at different structural levels due to shock excitation at rail level.

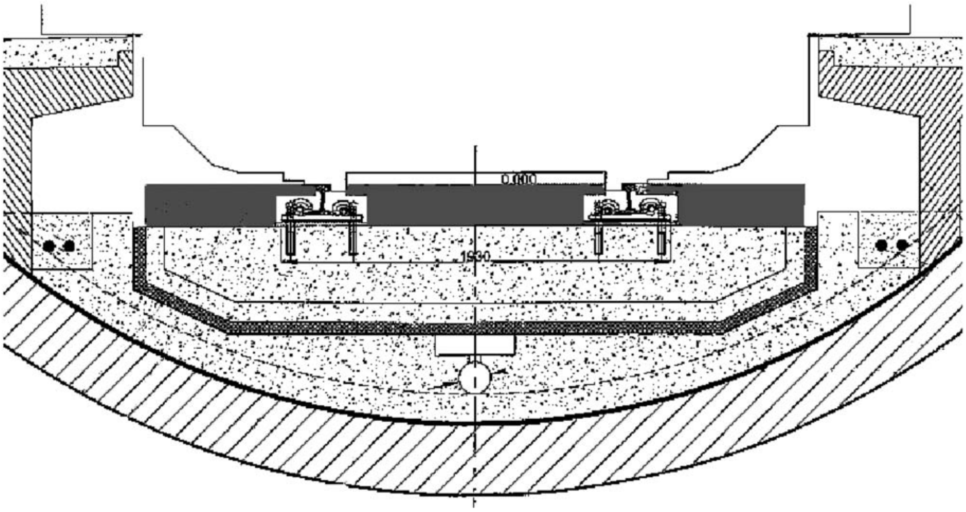

Fig. 13. Preliminary principal sketch of floating slab track in the drilled tunnel.

At the Viséstraat, bore–hole measurements were arranged in order to evaluate the damping as a function of distance for subsoil excitation sources. These were carried out in co-operation with the University of Liège. An air-gun was used which gives a very short pressure pulse. It was concluded that considerable vibration reduction could be obtained with increasing distance from the tunnel. However, this is the case only at higher frequencies. Ground absorption as a function of distance is strongly dependent on the wavelength. Only for frequencies above 30 Hz could a reduction of vibration level be observed for the properties close to the tunnel.

Based on the previous investigations regarding the excitation function, the train speed, the rail foundation, the tunnel and the transfer into the local environment, a numerical worksheet was created on which certain parameters can be adjusted rapidly and the impact observed.

It is clear that considerable vibration isolation in the tunnel will be required during its construction for those parts of the tunnel which are close to historic buildings. Therefore, an inertial mass is proposed for the rail foundation which will have high bending stiffness to spread the dynamic forces over larger areas (Fig. 13). In this way, the foundation stiffness can be reduced significantly resulting in limited forces on the tunnel lining and effective isolation at low frequencies.

From a dynamic point of view, foundation stiffness must be as small as possible. However, this is limited by the requirement for the maximum static deflection at the rail level. The static deflection at the rail level is approximately 2 mm for each individual axle load and the total deflection due to two adjacent bogies is approximately 5 mm.

The vibration velocities for the chosen parameters of the rail foundation have been computed for the rail, on the tunnel lining and in the basements and higher floors in the housing above the tunnel (Fig. 14).

The resonance frequency of the floating slab foundation is around 11 Hz. At this frequency vibration levels are amplified. The tunnel foundation provides attenuation above 50 Hz due to its inertia and stiffness. Soil is a very effective absorber at frequencies above 100 Hz. Structural

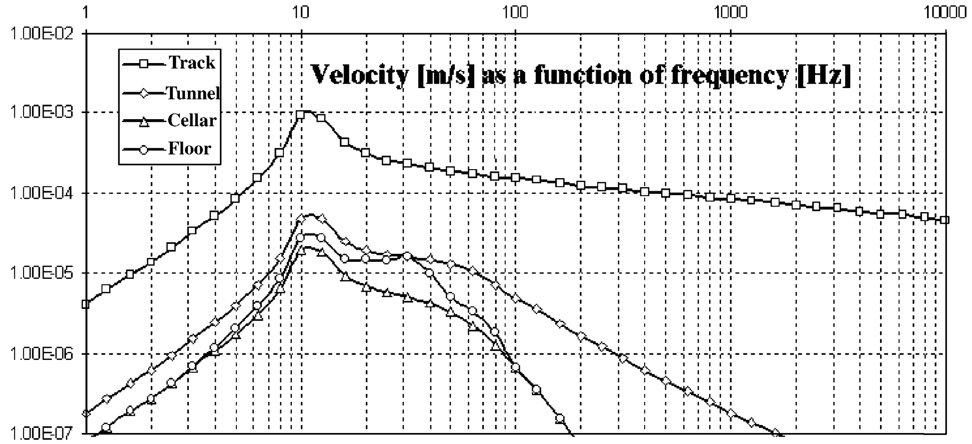

Fig. 14. Predicted vibration velocities as a function of frequency at different structural levels.

Table 1
Predicted vibration velocities at different structural levels

Vibration velocity	RMS (mm/s)			Peak (mm/s)			KB_{max}		
Excitation function	Min.	Avg.	Max.	Min.	Avg.	Max.	Min.	Avg.	Max.
Rail	0.94	1.57	2.46	3.74	6.28	9.83	2.65	4.44	6.95
Tunnel	0.05	0.08	0.13	0.19	0.33	0.52	0.13	0.23	0.37
Basement	0.02	0.03	0.05	0.07	0.12	0.20	0.05	0.09	0.14
Floor	0.03	0.05	0.08	0.12	0.20	0.32	0.08	0.14	0.23

amplification on the floor in the houses is observed at a resonance frequency of 30 Hz. Table 1 shows the data for an individual train at a speed of 80 km h^{-1} and for three different types of excitation function (minimum, average and maximum).

It is possible for a representative room in a structure, depending on its dimensions and acoustical absorption, to translate these vibration values into sound, which is known as structure-borne radiated sound.

For a criterion for both the vibration level (i.e., 0.3 mm s^{-1}) and the sound pressure level (i.e., 45 dB(A)), and due to the lower sensitivity of the ear at lower frequencies, for frequencies below 40 Hz the vibration criterion is dominant and above 40 Hz the sound pressure level criterion is the more stringent one. Applied here to a train passage, the expected sound pressure level is shown in Fig. 15 in one-third octave bands.

Clearly, the maximum is determined by the levels in the 63 and 80 Hz bands. It is experienced as a low frequency rumbling noise. The "equivalent" vibration value KB_{Ftr}, according to the German standard, is generally the more stringent. It is calculated that a value of 0.05 will be obtained compared to the criterion of 0.07.

These results are obtained with an inertial concrete mass of 3.2 m wide and 0.5 m deep and having a static spring stiffness of 8 kN/mm/m.

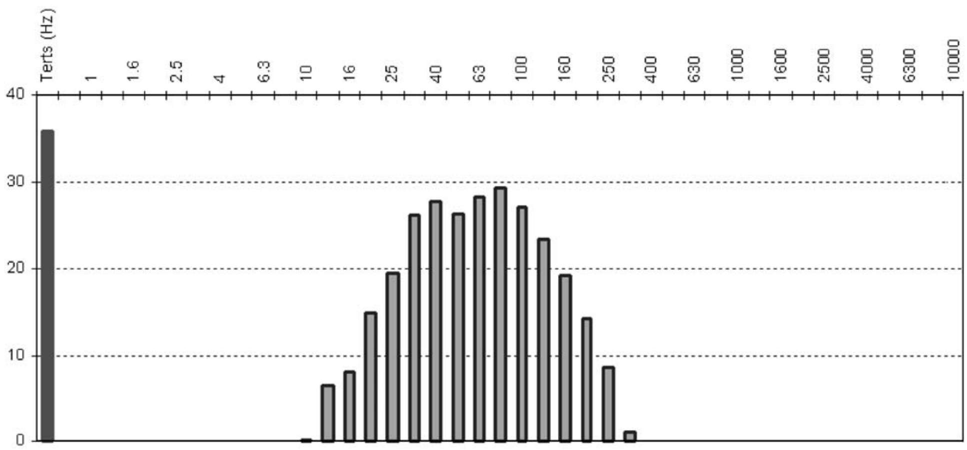

Fig. 15. Predicted A-weighted sound pressure level overall and in one-third octave bands in structure at Viséstraat.

In this project, contour maps were generated for several parameters, which took into account the distance between source and receiver.

The effect of the passage of a wheel flat on the local environment was also estimated. To date, a frequency response has been calculated, obtained by transforming the wheel flat into the frequency domain, calculating the result in the housing, and returning to the time domain again.

A wheel flat with length 4 cm (corresponding height is 0.4 mm) for a train speed of 80 km h^{-1} resulted in vibration levels of less then KB 0.1 and did not need to be considered further according to the German standard.

Above the proposed tunnel, several specific locations need attention. For instance at the Nachtegaelstraat there is an evacuation complex (and thus a potential mechanical short circuit due to these structures reaching the surface). At the Damplein the tunnel is close to the surface, and the tunnel is also close to the foundations of the Astrid Plaza Hotel. At these locations a qualitative comparison is done between tunnel–structure transfer functions with respect to those obtained at the Viséstraat. The transfer functions were computed by two-dimensional finite element models for the same boundary restrictions. The soil characteristics there were based on the same underground measured values as at the Viséstraat.

A generalized qualitative comparison based on one-third octave band differences in the transfer function between the source and receiver with reference to the Viséstraat location shows that the Viséstraat is indeed the most critical location. However, individual deviations do not differ by more than ± 5 to 6 dB at the dominant frequencies. Note that where the tunnel nears the surface at the Damplein a somewhat higher vibration level can be expected.

4. Conclusion

The study of the impact of sound and vibrations on the local environment for this railway project has been carried out through a combination of selected measurements and numerical modelling. They show the extent to which additional isolation is required.

The historic buildings and monuments on the Berchem–Antwerpen Central section are not likely to suffer damage due to vibrations. The vibration nuisance in the local environment is minimal because of the relatively large distance from the tunnel to the neighbouring houses. Limited isolation through the use of ballast mats is foreseen at the complexes near to track points and crossings.

It has been established that the additional vibration isolation in the tunnels must be installed at the track level using an inertial mass in order to ensure the reduction of vibration and sound levels.

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