THE INFLUENCE OF DEGRADATION OF SHEAR MODULUS OF SOIL ON PREDICTED RAILWAY VIBRATIONS IN TUNNELS EXCAVATED WITH TBM: A NUMERICAL ANALYSIS

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Abstract. In this paper the influence of the stress state within the soil on predicted railway vibrations in tunnels is studied, regarding Gs modulus degradation that occurs when the tunnel is excavated with Tunnel Boring Machine (TBM). The adopted procedure is uncoupled, using a 2D FEM model to simulate the process of tunnelling which incorporates the constitutive model known as Hardening Soil with small strain stiffness (HS-small). The Gs modulus values are then used in a dynamic numerical model based on a 2.5D FEM-PML approach in order to simulate the vibrations induced by railway traffic. Simulations are then performed assuming two different scenarios for the elastic properties of the ground: i) assuming Gs modulus for low strain deformation for whole domain, i.e., neglecting the perturbation of the soil due to the tunnel construction; ii) assuming values of Gs modulus computed taking into account the strains induced by the tunnel construction. The numerical results show a clear influence of tunnelling process, being more relevant at closest distances to the tunnel.
1. INTRODUCTION

Vibrations induced by railway traffic in tunnels is a special interest and development topic in recent years, mainly in urban areas where nuisance generated can be common if no preventive measures are taken or if the initial design is not right. It is reasonable to affirm that transport infrastructure in the 21st century has much more requirements than previously, since environmental aspects have gained a significant importance. Nowadays, the impact of vibrations on working and living environments is considered as an important problem of modern societies [1]. Moreover, the relevant growth of population around metropolitan areas implies the need for more sustainable underground transport systems. One of the key advances to minimize the impact of railway systems is the development of the numerical models that are capable of predicting correctly the level of vibrations on free-field or on buildings and to help in finding technical solutions for mitigating possible nuisances. For that, in last recent years several models have been developed in order to understand the complex problem of railway vibrations in tunnels. It is worth mentioning semi-analytical models such as those proposed by Hussein and Hunt [2], Kuo et. al [3] and Muller [4], whose main advantage is a high computational efficiency although they could not be rigorously applied in some situations. Complex numerical models have also been developed, such as those presented by Clouteau et al. [5] and Gupta et al. [6, 7], among others.

An interesting approach of the problem has been presented by Hung and Yang [8] and Lopes et al. [9, 10] in which 2.5D models are developed. The former is a 2.5D FEM-IEM model while the latter is a 2.5D FEM-PML model. The 2.5D models can be applied for longitudinally invariant structures, obtaining the 3D solution by numerical discretization of the cross-section combined with the Fourier transformation of the domain along the orthogonal direction. In addition to these three last researches, similar models have also been applied for the study of several railway lines both in surface and in tunnels [11-14].

In the above-mentioned studies the soil behaviour has been considered as linear-elastic as the vibrations induced by railway traffic in tunnels generate very small strains in soils. This approach seems to be clear and reasonably correct. Nevertheless, none of them has considered the possible effect of the tunnel construction on the degradation of the shear modulus of the soil. In fact and to authors’ knowledge, there are no studies about railway vibrations in tunnels in which this phenomenon had been taken into account. During tunnelling process the shear modulus of soil does not remain constant and its degradation magnitude depends on multiple factors such as diameter and depth of tunnel, excavation method and soil stiffness, among others. It is not easy to quantify standard values of shear strains during tunnel excavation although they could range between $10^{-4}$-$10^{-2}$ [15]. Even though tunnel excavation with TBM is a three dimensional problem, Panet and Guénot [16] demonstrated that 3D ground response can be approximately analysed with a plane strain approach, introducing a stress release coefficient that could be evaluated through 2D and 3D numerical comparisons. However, such comparisons are very limited in the technical literature. Because of underground railway vibrations is the main focus of this paper, a 2D simplified method has been applied for studying $G_s$ degradation during tunnelling process where the tunnel lining grouting has not been considered due to $G_s$ degradation induced by this process is very located in the soil attached to the tunnel lining.

Regarding the complete behaviour of the soil, from very small strains to large strains range, several constitutive models have been developed such as the Hyperbolic model [17], the Equivalent linear model [18], the Hardening soil model [19] and, recently, the Hardening soil model with small-strain stiffness [20], among others. The latter is applied in this paper in order to take into account $G_s$ degradation during tunnelling process.
2. NUMERICAL APPROACH

2.1 Generalities

The applied computational scheme is uncoupled, using a 2D FEM static model to simulate the tunnelling process and calculate $G_s$ degradation within soil by using the PLAXIS commercial code. The Hardening soil model with small-strain stiffness (HS-small) has been used to take into account the $G_s$ modulus degradation. Once calculated the resulting stress state in soil after tunnel construction, a 2.5D FEM-PML model formulated in the wavenumber/frequency domain is used to obtain the free-field response, assigning on every finite element the resulting soil stiffness from the 2D FEM static model. This stiffness (different for each finite element) remains constant during the computing process with the 2.5D FEM-PML model, since the strains induced by the traffic are very small. The computed results are compared with those obtained without regarding tunnel excavation, i.e. neglecting the perturbation of the soil due to tunnel construction. In the followings sections these numerical models are briefly described.

2.2 2D Numerical simulation of tunnelling with TBM

To simulate tunnel construction with TBM, a 2D FEM model has been used through software PLAXIS. A 2D simplified method is applied in order to calculate the stress state within soil. Some of the 2D available methods are: Core Support Method ($\alpha$-method), Lining Reduction Method ($\delta$-method) and Stress Reduction Method ($\beta$-method), among others [21]. In this research, $\beta$-method has been used to simulate the construction process of tunnel, whose main feature is to take into account arching effect that occurs within the soil due to near presence of tunnel face. The basis of this method is schematized in Fig. 1 and the idea is that the initial stresses $p_k$ that act in the contour of the tunnel are divided into two parts: the first is $(1-\beta)\cdot p_k$ that is applied to the unsupported tunnel and the second is $\beta\cdot p_k$ that is applied to the supported tunnel. In PLAXIS code this approach is performed through three construction phases, controlling in each one the staged construction process (through parameter $\sum M_{stage}$), as follows:

1- Phase 1: Generate the initial stress field with $K_0$-procedure in case of horizontal soil layers

2- Phase 2: Eliminate tunnel clusters applying an ultimate level of $\sum M_{stage}$ equal a $1-\beta$

3- Phase 3: Activate the lining, allowing a complete staged construction ($\sum M_{stage} = 1$)

The main uncertainty of this method applied to tunnel excavation is the $\beta$ value. This parameter can oscillate between 0.2-0.8 depending on excavation method, tunnel diameter, among others factors. In the TBM cases, $\beta$ value is usually between 0.4-0.8.

![Fig. 1. Schematic representation of the $\beta$-method for the analysis of tunnels](image-url)
2.3 2.5D FEM-PML approach for simulation of traffic vibrations

Contrarily to what happens on the tunnelling operations simulation, where a reasonable solution can be found using a 2D approach, the problem of vibrations induced by railway traffic is fully 3D and, consequently is not compatible at all with a 2D plane strain state approach. However, assuming that the system is invariant and infinitive along the longitudinal direction, the 3D solution can be achieved adopting a 2.5D procedure, i.e., only the cross-section of the problem needs to be discretised into finite elements (in the present case a finite elements approach is considered; alternative approaches using BEM or MFS can be found on [13, 22] since a Fourier transformation is applied regarding the longitudinal coordinate.

For the present case study, the 2.5D FEM approach seems to be advantageous since non-homogeneous cross-sections are considered. However, there is a pitfall when using finite elements approaches on dealing with dynamics of unbounded domains: the demands of limitation of the discretized domain gives rise to the need for treatment of the artificial boundaries in order to prevent the spurious reflexion of waves. In the present approach this requirement is fulfilled using a 2.5D PML’s approach along the artificial boundaries.

Due to the length limitation of the paper, the mathematical formalism inherent to the 2.5D FEM-PML approach is not here presented. However, it can be found in other papers of the authors, namely in: [9, 10, 23].

The prediction of vibrations due to railway traffic in tunnels constitutes a challenge by several reasons. One of these particular aspects is the train-track interaction, which is provided by the track irregularities, the out-of-roundness of the wheels or by the non-homogeneous support of the rail. These heterogeneities of the system give rise to inertial forces on the train, i.e., to a train-track interaction force which is variable along time. By that reason, the structural behaviour of the train needs to be also simulated on the prediction model. In the present model, a sub-structured approach is followed, where the train simulated by a multi-body approach and the remaining system by the 2.5D FEM-PML model described above. Fig. 2 shows a schematic representation of the distinct models considered on the sub-structured approach. The dynamic interaction of the different models is also depicted on the same figure. Once again, the limitation of length of the paper prevents an exhaustive description of the train-track interaction model, but the reader can found a detailed description on previous papers of the authors, namely in: [9, 10, 24].

![Fig. 2. Schematic representation of the sub-structured model used for prediction of vibration due to railway traffic](image-url)
Despite the ability of the model for dealing with different sources of excitation of the train, as for instance the out-of-roundness of the wheels, in the present application it is assumed that the only source of dynamic excitation corresponds to the track unevenness.

3. APPLICATION EXAMPLE

3.1. Objectives

As emphasized above, the main objective of this research is to study the influence of $G_s$ degradation that occurs within the soil during the excavation process of tunnel with TBM on predicted railway vibrations in tunnels. For that, an only case has been analysed with a homogeneous soil for the whole domain, where $\beta$-value has been considered as 0.5.

3.2. Model description

*Fig. 2* shows the geometry of the finite element mesh used to simulate the tunnel construction with PLAXIS code. Although it usually works with unstructured mesh, in this case it has been structured in order to use a similar mesh to study both tunnel construction and railway vibrations. The size of mesh to analyse the tunnel excavation has been estimated according to Meissner [24]. The depth of tunnel has been chosen in order to consider a realistic geometry in which is possible to build it with TBM method.

*Fig. 4.* shows the 2.5D FE-PML model to study railway vibrations. The size of finite element in the soil has been estimate regarding 6 elements per wavelength, limiting frequencies up to 80 Hz. The interest domain is limited by PML’s in order to avoid the spurious reflexion of the waves that impinges the artificial boundaries.

3.3. Soil and tunnel properties

The soil constitutive model applied is the *HS-small* (only in 2D FEM model for simulating tunnel construction) which is based on the *Hardening soil model* but considering the complete stress-strain curve including the very small strain range. The *Hardening soil model* and the *Hardening soil model with small-strain stiffness* are explained in detail by Schanz et al. [19] and Benz [20] respectively. The soil properties are shown in table 1 according to the *HS-small model.*

![Finite element mesh (in metres) for tunnelling](image)
The relationship between $E_{50}^{\text{ref}}$ and $E_{oed}^{\text{ref}}$ has been estimated according to Schanz [25] and the relationship between $E_{50}^{\text{ref}}$ and $E_{ur}^{\text{ref}}$ is supposed to be equal to 2. $G_0^{\text{ref}}$ has been calculated as follows:

$$G_0^{\text{ref}} = \frac{E_0^{\text{ref}}}{2(1+\nu_{ur})} \quad (1)$$

Where $E_0^{\text{ref}}$ has been calculated according to Alpan [26]. The relationships given by Alpan between $E_0$ and $E_{ur}$ are shown in Fig. 5.
The “m” exponent has been estimated according to Benz [19]. The soil has been considered as normally consolidated and in this way $K_0$ has been calculated according to Jaky [27]. Although $\gamma_{0.7}$ is dependent on confining stress, in this case it has been considered to be constant for the whole domain and is obtained as follows [20]:

$$\gamma_{0.7} = \frac{0.385}{4G_0} \left[ 2c' (1 + \cos(2\varphi')) + \sigma'_1 (1 + K_o) \sin (2\varphi') \right]$$

(2)

Where $\sigma'_1$ is the effective vertical stress in the middle of considered soil layer in the numerical model. Fig. 6 shows the $G_s/G_0-\gamma_s$ relationship for the soil in the HS-small constitutive model and the tunnel properties are summarized in Table 2.

**Table 2: Tunnel properties**

<table>
<thead>
<tr>
<th>Property</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Thickness (m)</td>
<td>0.35</td>
</tr>
<tr>
<td>Inner radius (m)</td>
<td>3.9</td>
</tr>
<tr>
<td>Density (kg/m³)</td>
<td>2500</td>
</tr>
<tr>
<td>Elastic modulus (kN/m²)</td>
<td>$30 \cdot 10^6$</td>
</tr>
<tr>
<td>Poisson ratio</td>
<td>0.2</td>
</tr>
</tbody>
</table>

**3.4. Stress state within soil after tunnelling**

Fig. 7 shows computed $G_s/G_0$ and shear strains ($\gamma$) contours after numerical simulating of tunnel construction, in which can be seen as Gs degradation is more evident closed to lining, even reaching
the ground surface although the effect of tunnelling process at ground surface is limited until a
distance of 25-30 meters from symmetric axis of the model.

![Computed contours Gs/Go (left) and shear strains (γ) (right)](image)

3.5. Track and train properties

In what concerns the modelling of vibrations induced by railway traffic, additional description of
the model is required. Two distinct computations are performed: i) non disturbed ground; ii)
disturbed ground due to tunnel construction operations. In the former case, the shear modulus of the
ground is assumed constant with value of G₀ indicated in Table 1. On the other hand, the second
computation set corresponds to the disturbed ground, where the distribution of G depicted in Fig. 7
is adopted. In both situations, the volumetric mass was assumed as 2000 kg/m³; the Poisson ratio
equal to 0.2 and a hysteretic damping ratio of 0.04.

The railway track is composed by a continuous concrete slab track 0.3 m thick and 2.5 m wide,
with a longitudinal bending stiffness of $1.62 \times 10^8$ N/m² and a mass of 3000 kg per unit of length.
The rails, materialized by UIC60 profiles are continuously supported by railpads with a stiffness of
$2.5 \times 10^8$ N/m² and a damping coefficient of $6 \times 10^4$ Ns/m². It is assumed a floating slab solution,
introducing a resilient mat between the slab and the tunnel invert. The stiffness of the mat is
$0.153 \times 10^9$ N/m² per meter in the longitudinal direction and a damping of $5.5 \times 10^4$ Ns/m² is
considered.

In what concerns to the rolling stock, it was assumed the passage of the Alfa-Pendular train at a
running speed of 40 m/s. The main geometrical and mechanical properties of the train Alfa-
Pendular are summarized in Fig. 8 and in Table 3.

![Alfa-Pendular geometry](image)

| Axles Mw (kg) | 1538-1884 |
| Primary Kp (kN/m) | 3420 |
| suspension Cp (kNs/m) | 36 |
| Bogies Mb (kg) | 4712-4932 |
| Jb (kg/m²) | 5000-5150 |
| Car body Mc (kg) | 32900-35710 |

Table 3: Mechanical properties of the train
As mentioned above, the first step of the calculation procedure concerns the solving of the train-track interaction problem. The quasi-static and dynamic excitation mechanisms are uncoupled, being the solution obtained through a superposition procedure.

In the present study, the source of vibration is due to the track unevenness. An artificial unevenness profile was generated taking into account the power spectral density of amplitude of the track unevenness for a range of wavelengths between 28 m and 0.55 m. The following equation was used for address the PSD of the track unevenness:

\[
S(k_1) = S(k_{1,0}) \left( \frac{k_1}{k_{1,0}} \right)^{w}
\]

where, \( k_{1,0} = 1 \text{ rad/s} \), \( w = 3.5 \) and \( S(k_{1,0}) \) was assumed equal to \( 1 \times 10^{-8} \text{ m}^3 \).

Once the train speed was assumed to be 40 m/s, the unevenness profile considered excitates the train in the frequency range between 1.4 Hz to 72 Hz.

4. RESULTS AND DISCUSSION

4.1. Transfer functions for stand still loads

Firstly, to the study of the impact of construction operations of the tunnel on the vibration levels perceived at the ground surface, a simple analysis is presented where a unitary stand still load is applied at the centre of tunnel invert and the dynamic response is recorded at different positions at the ground surface. This simple analysis allows observing how the shear modulus degradation affects the transfer functions between the tunnel invert and the ground surface. In the present case it is assumed that the load is applied at the cross section defined by \( x = 0 \text{ m} \) (\( x \) is the longitudinal axle), and the receiver points are located at the same cross-section.

Fig. 9. Vertical displacement transfer functions due to a load applied at the tunnel invert and evaluated at distinct positions of ground surface: a) P1(0,0,0); b) P2 (0,5,0); c) P3 (0,10,0); d) P4 (0,15,0); e) P5 (0,20,0) f) P6 (0,40,0) (blue line – non disturbed soil; red line – disturbed soil)
The analysis of Fig. 9 allows to see an agreement of the response obtained for both scenarios in the lower frequency range, i.e., up to 15 Hz. Actually, the wavelengths generated in this frequency range are quite large, being not so much affected by point variations of the ground stiffness. On the other hand, for the receiver points located close to the tunnel, and for frequencies above the mentioned limit, a relevant influence of the construction operations on the transfer function is detectable. On Fig. 9.a to d, it is possible to see a change of the frequency of the troughs of the transfer function when the stiffness degradation due to the tunnel construction is taken into account. On the other hand, it seems that the difference between the two scenarios evanesces with the increase of the distance between the source and the receiver. This aspect is quite evident in Fig. 9f, where, for frequencies up to 50 Hz, there is a good agreement between the results obtained from both scenarios. However, for frequencies above this limit, differences remain, being justified by the fact that the wavelengths generated are short enough to be influenced by local changes on soil stiffness.

4.2. Vibrations due to railway traffic

Figs. 10 and 11 show time record and one-third octave spectrum of vertical velocity due to passage of the Alfa Pendular train respectively. It is worth mentioning that only 4 points at ground surface are shown, located at several distances from symmetric axis of the model: 0, 5, 20 and 40 metres.

![Fig. 10. Time record of vertical velocity at distinct points located at the ground surface: a) P1(0,0,0); b) P2 (0,5,0); c) P5 (0,20,0) d) P6 (0,40,0) (blue line – non disturbed soil; red line – disturbed soil)](image)

Regarding the compared response in the time domain is noteworthy that the effect of taking into account the tunnelling process is more notorious at points located closer to the tunnel, resulting minor differences between the two scenarios with the increase of the distance to the tunnel. In fact, the difference found at a distance of 40 metres is almost negligible. However, at the other points (0,
5 and 20 metres) the differences are noteworthy, being more important for smaller distances to the tunnel.

A more clear comparison between the two considered scenarios is provided by Fig. 11, in which it can be seen that the influence of tunnelling process is more evident with the decrease of the distance between the source and the receiver. However, regarding one-third octave spectrum it is also possible to comment that there are more differences in frequencies above 15 Hz than below this value, due to the wavelengths generated below 15 Hz are quite large, being not so much affected by point variations of the ground stiffness. Similar conclusion was also found for the analysis of transfer functions depicted on Fig. 9. Comparing the two scenarios in terms of decibels, it is possible to conclude that in frequencies below 15-20 Hz the maximum difference is 3 dB while above 20 Hz it is possible to achieve differences around 7 dB for the points located close to the tunnel. However, the influence of tunnelling process evanesces with the increase of the distance source-receiver. Actually, the differences found on Fig. 11d are almost negligible.

5. CONCLUSIONS

In this paper, the influence of the stress state within the soil on predicted railway vibrations in tunnels has been studied, regarding the degradation of shear modulus that occurs when the tunnel is excavated with Tunnel Boring Machine (TBM). In this way, the main conclusions are:

- The influence of the degradation of shear modulus of soil during tunnelling process with TBM on predicted railway vibrations can be relevant, providing a higher level of vibrations at the ground surface when $G_s$ degradation is considered.
- This fact is more notorious at the closest points of the tunnel, being practically negligible at distances greater than 20 metres from the line of centre of the tunnel.
The influence is more important for frequencies above 15 Hz than below this value. So, the maximum difference above this mentioned limit has been 7 dB while for frequencies up to 15 Hz has been 3 dB.

The numerical results show that the effect of degradation of shear modulus of the soil during tunnelling process on predicted railway vibrations should be taken into account in design process in order to find more accurate results.

The research is on-going and numerical studies will be performed for other geotechnical conditions in order to accomplish more conclusive remarks.

REFERENCES


