Reduction of train-induced building vibrations by using open and filled trenches

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Abstract

A numerical investigation on the effectiveness of open and in-filled trenches in reducing the building vibrations due to passing trains is presented. Particularly, a two-dimensional soil-structure system containing the cross-section of a railway embankment, the underlying soil, a trench barrier and a nearby six-storey building is considered. For the analysis, a time domain coupled boundary element-finite element algorithm is employed. Unlike most of the previous formulations, this model completely considers the soil-structure interaction effects and directly determines the effect of the wave barrier on the structural response. The effects of geometrical and material properties of the trench and its backfill material on the structural response are investigated. The results point out that using a trench barrier, a reduction level up to 80% of the building vibrations and internal forces can be achieved. Increasing the depth or the width of a trench may improve its reduction effect and a softer backfill material results in a better isolation effect.

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1. Introduction

Due to train traffic, machine operations, pile driving, or blasting, ground vibrations are generated that may cause distress to adjacent structures and annoyance to residents. For instance, most of the vibration energy generated due to a train passage is carried by Rayleigh waves that propagate close to the soil surface and transmit the vibrations to the structures via their foundations. These vibrations lie in the frequency range of 4–50Hz and may bring some structures to resonance with their vertical modes [1–3]. This type of vibrations can be a major problem in densely populated areas and for structures, which are housing sensitive machinery. Therefore, in many countries new environmental regulations have been introduced placing some constraints on railway operations. Consequently, the isolation of the traffic-induced vibrations has become an important issue in recent years.

Generally, it is possible to prevent the adverse effects of these vibrations by providing a suitable wave barrier between the source and the structure to be protected. This system of vibration isolation can be classified into
two categories, source isolation and receiver isolation. With source isolation barrier it is tried to reduce the vibrations at their source. It should be installed surrounding the vibration source or at close distance to it. The receiver isolation barrier, on the other hand, usually is built away from the source, surrounding the structure to be protected. Stationary sources of vibration, e.g. machines working with a certain frequency, can be effectively isolated by a source isolation barrier, whereas a receiver isolation barrier is effective for a wide variety of wave generating sources.

Different types of wave barriers, varying from very stiff concrete walls or piles to very flexible gas cushions, are discussed in [4–7]. Among them, both, open and in-filled trenches are the most common in practical application since they present effective and low cost isolation measures.

The published literature reveals that in early studies, efforts have been directed mainly towards analytical studies and some experimental works to investigate the problem of isolation by means of trench barriers. Only a few experimental studies present some design guidelines for particular cases, but they are rather limited in their scope. Woods [8] and Haupt [9,10], for instance, conducted a series of field tests, analytical studies and laboratory model experiments in order to study the screening performance of open trenches and concrete walls. Moreover, the wave diffraction by spherical and parabolic obstacles has been studied analytically by some researchers [11,12]. However, the closed form solutions were confined to simple geometries and idealized conditions.

For this reason, various numerical methods were used by many authors for solving wave propagation and vibration reduction problems. For example, Lysmer and Waas [13] employed the lumped mass method, while Segol et al. [14] applied finite elements along with special non-reflecting boundaries to investigate the isolation efficiency of open and bentonite-slurry-filled trenches in a layered soil. Frequently, also the finite difference technique was used to study the scattering of a Rayleigh wavelet by a rectangular open trench [15]. Nevertheless, an underlying bedrock has to be inevitably included in the analysis models along with some sort of artificial transmitting boundaries due to the numerical constraints of the above mentioned methods.

In the last two decades, the boundary element method (BEM) has been applied for a significant portion of the studies on wave propagation problems. In particular, this method is very well suited to investigate the wave propagation in soils, since the radiation into the ground (e.g. into a halfspace) is directly included in the formulation. Many authors adopted the BEM for the analysis of isolation effects of open and in-filled trenches, investigating different types of soils. Thus Emad and Manolis [16] considered shallow open trenches of semi-circular and rectangular shape in a two-dimensional soil profile, while Beskos et al. [17] and Leung et al. [18,19] investigated open and filled trenches in homogeneous and non-homogeneous soil, respectively. Ahmad and Al-Hussaini [20] and Al-Hussaini and Ahmad [21] concentrated on simplified design methodologies for wave barriers and vibration screens, also looking at an active isolation of machine foundations by open trenches [22]. Trenches filled with different materials are discussed in [23].

In all cases, however, it can be observed that the boundary element method is not well suited for the modeling of irregular geometries or a possible non-linear material behavior of soft soils and/or structural foundations. To overcome this drawback, several procedures for coupling finite with boundary elements or finite with infinite elements have been proposed, following the pioneering work of Zienkiewicz and his co-workers, who developed a FEM/BEM coupling scheme [24] as well as a combination of the FEM with infinite elements [25]. Thus von Estorff and Prabucki [26], for instance, used a FEM/BEM scheme for dynamic soil-structure interaction including trench problems, while Yang et al. [27,28] concentrated on a coupling of finite and infinite elements applied to study the reduction of train induced vibrations using different types of wave barriers.

With the exception of a few, most of the previous researches have mainly dealt with the development of different numerical methodologies as a tool for the analysis of vibration isolation problems. Parametric studies have been rather limited and mostly performed in the frequency domain. Moreover, the reduction of the nearby ground surface vibration amplitude was the major concern.

In this paper, the investigation is focused on the effects of using trench barriers for the reduction of nearby building responses through a parametric study directly in the time domain. The building is directly considered in the mathematical modeling and analysis. The coupled boundary element–finite element (BE–FE) algorithm developed earlier by Adam et al. [29] is extended to handle the building structure as frame elements [30,31] and employed for the numerical analysis of the current problem, some details are given in Appendix A. Therefore, different from others, the soil-structure interaction effect is automatically taken into account and the effect of the barrier on the structural response is obtained directly. The response of the building is given in terms of accelerations and internal forces, which represent the most important design parameters for structural engineers.

2. Numerical model and considered parameters

A reinforced concrete six-storey building frame, as shown in Fig. 1, shall be considered in detail. Its width
is 12m and its height is 18m. The building is located to the right hand side of a railway embankment. In the direction perpendicular to the considered two-dimensional profile, the spacing between the frames is assumed to be 4m. The distance from the centerline of the track to the left hand side of the building is 20m. The building foundation level is located at a depth of 1.5m below the soil surface. Two alternative types of foundations are assumed. The first type consists of strip footings of 0.8m thickness. The second type is a raft foundation of the same thickness. All elements of the building frame are assumed to have a uniform cross-section of 0.30m breadth and of 0.60m thickness.

The top surface of the railway embankment is located at 1.5m above the soil surface of a homogeneous half-space. The material properties of the halfspace, the embankment layer, the foundation, and the frame elements are given in Table 1. A trench barrier of width $W$ and depth $D$ is assumed to be located at a distance $L$ measured from the centerline of the trench to the left hand side of the building. The trench width $W$, the depth $D$ and the distance $L$ are assumed to be variable parameters.

The soil-structure system shown in Fig. 1 is divided into a BEM and a FEM subsystem. The BEM subsystem, modeled by boundary elements, is a domain representing the uniform part of the underlying soil (halfspace). The FEM subsystem is divided into two parts. The first part is a domain including the wave source, i.e., the train track, and a certain part of the underlying soil containing the trench, the building foundation and any sort of soil irregularity. This region is discretized by means of plane strain solid elements. The finite element mesh is shown in Fig. 1. The second part consists of the building frame modeled using plain frame elements [30]. Since these have three degrees of freedom at each node, special care must be taken when coupling them with the solid elements of the foundation, which have only two translational degrees of freedom per node. Therefore, the previously developed time domain BE-FE algorithm [29] needed to be modified to deal with the current problem. Herein, the rotational degrees of freedom at the connecting nodes are condensed out during the solution of the governing equation of motion of the coupled system.

<table>
<thead>
<tr>
<th>Material</th>
<th>Mass density $\rho$ (t/m$^3$)</th>
<th>Shear wave speed $V$ (m/s)</th>
<th>Poisson’s ratio $v$ (-)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Halfspace</td>
<td>2.00</td>
<td>0150</td>
<td>0.33</td>
</tr>
<tr>
<td>Embankment</td>
<td>2.00</td>
<td>0250</td>
<td>0.33</td>
</tr>
<tr>
<td>Concrete foundation</td>
<td>2.50</td>
<td>2400</td>
<td>0.20</td>
</tr>
<tr>
<td>Columns and Girders</td>
<td>2.50</td>
<td>2400</td>
<td>0.20</td>
</tr>
<tr>
<td>Soil-bentonite mixture</td>
<td>Variable (1.2, 1.6, 2.0)</td>
<td>Variable (30, 60, 90)</td>
<td>Variable (0.25, 0.33, 0.45)</td>
</tr>
</tbody>
</table>

Fig. 1. Numerical model of the considered soil-structure system and the time history of the applied load.
system. Later on, after the solution at each time step, they are recovered. Such a dynamic condensation procedure is straightforward and can be found in detail, for instance, in [30].

Two concentrated line loads in a horizontal distance of 1.8 m to each other are applied simultaneously to represent the trainload. The time history of each load consists of four consecutive impulses; each impulse having a time duration of 0.02 s and 1000 kN in amplitude. The time between each two consecutive impulses is 0.02 s as shown in Fig. 1. This load combination posses a wide frequency contents with a predominate range from 20 to 35 Hz, and a central frequency of about 27 Hz [31]. This covers the frequency range that is expected to be caused by a heavy axle train passage [1,2]. In the analysis, a total time period of 0.75 s is considered which is divided into 600 time steps with a duration of 0.00125 s each.

Since most of the previous investigations have dealt with a harmonic load of a single frequency, it was common to relate each of the abovementioned parameters to the Rayleigh wavelength \( L_r \), which depends on the applied frequency and the soil properties. For example, some researchers propose the trench depth to vary between 0.6 \( L_r \) and 1.33 \( L_r \), and the trench width to be built between 0.1 \( L_r \) and 0.5 \( L_r \) [8–10,18–22]. The trainload applied in the current investigation, however, covers a wide frequency range. Therefore, the values of the studied parameters need to be considered in an average sense in order to cover the predominant frequency range given in Table 2.

In the case of the in-filled trenches, the backfill material is a soil–bentonite mixture which is assumed to be much softer than the natural soil. The mass density \( \rho \) of the backfill material, its Poisson’s ratio \( v \), and its shear wave speed \( V \) are also considered to be variable parameters. The investigated values are summarized in Table 2.

### Table 2
Geometric parameters of the trench barrier

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Assumed values (m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Distance from building left side (( L ))</td>
<td>2.0, 3.0, 5.0, 8.0</td>
</tr>
<tr>
<td>Depth of trench below ground surface (( D ))</td>
<td>3.0, 4.50, 6.0, 8.0</td>
</tr>
<tr>
<td>Width of trench (( W ))</td>
<td>0.50, 1.0, 1.50, 2.0</td>
</tr>
</tbody>
</table>

3. Analysis of the structural response

First, the analysis is performed to determine the building response due to the applied trainloads without any trench barriers. The building frame is supposed to be subjected to the usual dwelling static loads, which are not considered in this analysis. Only the structural response due to dynamic loads will be determined.

Each floor mass is assumed to be attached to the floor girder to account for their inertial effects during the dynamic analysis. The case of a building resting on strip footings is considered to be the reference case. The building response is obtained in terms of internal axial force, shear force and bending moments. The left hand side column C1, the left intermediate column C2, and the first floor girder G1 are selected for the discussion. In addition, the acceleration is depicted on the selected points A, B, C and E. All details of the geometry can be found in Fig. 1.

The absolute maximum values of the building response are summarized in Table 3. The overall maximum axial force occurs in the left hand side column C1 while the maximum shearing force and the maximum bending moments can be observed in the intermediate column C2. It is interesting to point out that the resulting maximum dynamic axial forces in the columns represent about 30% of the expected static service load on column C1 and about 10% of the static service load on column C2. Moreover, the resulting maximum dynamic bending moment in the girder G1 is comparable to the expected maximum static service bending moments. Since this moment is reversible, the bending action on critical sections of the girder could be doubled due to the train passage. Concerning the vibrations, the maximum vertical acceleration occurs at the upper storey. As expected, the maximum values of the accelerations are gradually decreasing towards the lower levels of the building. The lowest value can be observed in the

### Table 3
Dynamic structural responses in case of using strip footings or raft foundations

<table>
<thead>
<tr>
<th>Location</th>
<th>Action</th>
<th>Strip footings</th>
<th>Raft foundation</th>
<th>% Reduction</th>
</tr>
</thead>
<tbody>
<tr>
<td>Column C1</td>
<td>( N )</td>
<td>202.24</td>
<td>187.54</td>
<td>7.26</td>
</tr>
<tr>
<td></td>
<td>( Q )</td>
<td>29.80</td>
<td>24.69</td>
<td>17.10</td>
</tr>
<tr>
<td></td>
<td>( M )</td>
<td>66.09</td>
<td>54.78</td>
<td>17.11</td>
</tr>
<tr>
<td>Column C2</td>
<td>( N )</td>
<td>98.42</td>
<td>19.91</td>
<td>79.77</td>
</tr>
<tr>
<td></td>
<td>( Q )</td>
<td>32.56</td>
<td>30.31</td>
<td>6.91</td>
</tr>
<tr>
<td></td>
<td>( M )</td>
<td>72.22</td>
<td>67.23</td>
<td>6.90</td>
</tr>
<tr>
<td>Girder G1</td>
<td>( N )</td>
<td>27.73</td>
<td>19.00</td>
<td>31.48</td>
</tr>
<tr>
<td></td>
<td>( Q )</td>
<td>32.48</td>
<td>27.29</td>
<td>15.98</td>
</tr>
<tr>
<td></td>
<td>( M )</td>
<td>69.19</td>
<td>58.12</td>
<td>15.99</td>
</tr>
</tbody>
</table>

\( ^a \) \( N \) = Axial force (kN).
\( ^b \) \( Q \) = Shearing force (kN).
\( ^c \) \( M \) = Bending moment (kNm).
first storey (see Table 3), which indicates that during a train passage, the residents living on the upper floors will suffer the vertical vibration more than those on the lower floors. To find out the effect of the foundation type on the dynamic behavior of the structure, the analysis is performed assuming that the building is constructed on a raft foundation without any trench barrier. The resulting response and the achieved reduction as a percentage of the corresponding values in case of using strip footings are also given in Table 3.

It is very interesting to observe, that the use of a raft foundation greatly reduces the axial force in column C2: A reduction of up to 80% with respect to the original value is ascertainable. Only a slight reduction effect of about 7% occurs in the axial force in column C1 and in the bending moments in column C2. A reasonable reduction effect of about 16–30% can be observed in the case of the internal forces in the girder G1, of the shearing force, and of the bending moments in column C1. Concerning the vertical accelerations of the floors, the use of a raft foundation has reduced the maximum values by 15–25% as given in Table 3. Therefore, it can be stated that for buildings close to railway tracks, a raft foundation is preferable and better than the strip footings. However, additional costs need to be taken into account and compared with the costs of other types of reduction measures.

4. Parametric study of the open trench

The analysis is performed for the structure with strip footings considering the existence of an open trench as shown in Fig. 1. The geometric variables are varied as given in Table 2. Note that when a certain parameter is investigated, the other two parameters are kept constant. Therefore, unless otherwise specified, the depth \( D \) of the trench and its width \( W \) are taken as 4.5m and 0.5m, respectively. The distance \( L \) between the trench and the building is assumed to be 2.0m. Since the reduction in the bending moment is identical with the reduction in the shearing forces, only the axial and shearing forces will be discussed in what follows.

4.1. Effect of the trench depth

For the purpose of this investigation, it is assumed that the depth of the open trench varies between 3m and 8m. It should be mentioned, however, that the open trench can be considered as a limit situation, since in reality—depending on the soil condition—its vertical sides will need special supporting measures when exceeding a certain depth.

In Fig. 2, the resulting reduction of the maximum internal forces for different values of the trench depth is depicted. As expected, it can be observed that for all internal forces the achieved reduction is getting larger with an increasing depth of the trench. Generally, the reduction in the column axial force, which is more related to the vertical vibration, is larger than in the shearing forces and the bending moment. In addition, the reduction in the columns axial forces is larger than the reduction in the girder axial force. The axial force in column C1 achieves a reduction of 35% for a trench depth of 3m and 80% for a depth of 8m. The axial force in column C2 shows a reduction of 50% and 80% for the same depths, respectively. The other forces get reduction values in the range of 10–50%. When the trench depth is doubled from 3m to 6m, the reduction value is approximately doubled for all forces.

In Fig. 3, the resulting time history of the vertical accelerations at point A is shown, assuming no trench as well as a trench of 3m and 6m depth. The figure indicates a considerable reduction in the acceleration amplitudes that achieve about 50% reduction in the case of 3m depth and about 75% reduction in the case of 6m depth. Moreover, it can be observed that there is a time delay in the wave arrival at the structure, and the vibration possesses a different waveform for each case. The time delay means that in the presence of a trench the waves travel a longer distance surrounding the trench. The altered waveform indicates that some sort of wave interference and transformations have occurred along.
The wave path [28]. The vibrations at the other selected points showed similar trends. Therefore, the details shall be omitted here. Also, it should be mentioned that using a raft foundation as discussed in the previous section, the reduction levels in forces and accelerations are lower than the ones mentioned here for the strip footings.

Based on the above findings, one can state that the open trench reflects a portion of the surface Rayleigh waves and enforce the other body waves to travel vertically downwards inside the soil leading to a longer wave path and more attenuation for the waves before hitting the foundation of the structure. This causes a considerable reduction in the wave amplitude and changes the waveform of the transmitted vibrations, especially in the vertical direction. For deeper trenches, this effect becomes more pronounced and the reduction in the internal forces increases.

4.2. Effect of the trench width

Fig. 4 shows the resulting reduction in the internal forces for different values of the trench width. The increase of the trench width results in an increase of the achieved reduction. Similar to the trench depth effect, the reduction in the axial force is larger than the reduction in the shearing force and bending moment. The obtained reduction in column C1 is about 55% for a width of 0.5 m, and about 70% for a width of 2 m. This means when the excavated trench volume is four times larger, the achieved reduction is increasing by about 25% only. Besides, the reduction ratios in the axial force in column C2 and girder G1 get slightly higher with an increasing width. Thus, it can be stated that increasing the depth of a trench is more effective than changing its width.

In Fig. 5 the time histories of the vertical acceleration at point A are depicted. The cases of a 0.5 m and a 1.5 m trench are compared with the acceleration without a trench. In fact, only little differences in the acceleration amplitudes can be noticed in the two cases of using a trench. Almost similar waveforms with no time delay occur which indicates that increasing the trench width has only little effect on the vibration and its reduction. Therefore, it is advisable to perform a feasibility study for each particular case to achieve the optimum width and depth of a trench resulting in a maximum reduction of the response with the lowest cost.

4.3. Effect of the distance between the trench and the building

The internal force reduction obtained by the open trench located at different distances $L$ from the building is depicted in Fig. 6. As a general trend, for all forces the reduction value decreases with an increasing distance,
except for the normal force in column C2. Moreover, when the distance becomes more than 4m (column C1) and 5m (column C2), the shearing forces and the bending moments suffer an adverse effect and their values are amplified as indicated by the negative sign of the reduction in Fig. 6(b). These internal forces are more related to the horizontal motion which mainly results from the scattered waves in such cases where vertical loads are applied.

The horizontal acceleration time histories and their frequency contents resulting at point A for a distance of 2m and 8m are shown in Fig. 7(a) and (b), respectively. Again, the result without a trench is included in the comparison. It can be noticed that the acceleration amplitude in the case of 8m distance is larger than the amplitude in case of 2m distance with elongated or stretched wave cycles. Although the maximum amplitude of the acceleration in the case of 8m distance is smaller than the maximum amplitude in the case of no trench, it has a smaller number of cycles for the same elapsed time. Moreover, Fig. 7(b) implies that the frequency content amplitudes of the horizontal vibration...
in the case of 8 m distance is amplified in the lower frequency range (2–10 Hz) which might be closer to the eigenfrequencies of the structure leading to a higher structural response. This would explain the observed amplification in the shearing force and bending moment. Another possible reason is that the scattered waves caused more different horizontal motions at the foundation level leads to higher internal forces.

Based on the above observations, one can state that the relatively long distance between the trench and the building give more chances to the occurrence of wave interferences between the body and the surface waves. This may lead to a re-amplification of the previously reduced waves and to a generation of more scattered horizontal waves. It should be mentioned that in the case of source isolation, the distance between the trench and the building might be longer than in the present case. On the other hand, the trench barrier gets closer to the source and therefore might be more effective than concluded by many researchers [22, 23, 28].

Thus, one can roughly divide the distance between the wave source and the building into three zones. The first zone is close to the building where the receiver isolation using an open trench could be more effective. The second zone is the one close to the source where the source isolation using an open trench could be more advantageous. Finally, the third zone is the intermediate distance between the first and second zone. Here the isolation using an open trench is not very efficient and may even lead to an adverse effect, i.e., an amplification of the structural response. The extent of each zone depends on the soil conditions, the type of the structure, and the nature of the applied load.

5. Parametric study of the in-filled trench

In this section, the analysis is performed assuming that the trench barrier is filled with a bentonite–soil mixture, which is softer than the natural soil. Logically, a softer barrier is expected to approach the condition of an open trench as the backfill material is made softer and softer. The advantage of an in-filled trench is that one can achieve larger trench depths with no need for permanent lateral supports of the vertical sides.

Throughout the following studies, the mass density, the shear wave speed and the Poisson’s ratio of the backfill material are introduced as new variable parameters in addition to the previously mentioned geometric parameters. In order to relate the shear wave speed ratio between the backfill material and the original soil to a more practical parameter, the impedance ratio (IR) that is frequently used in geotechnical engineering is introduced here as:

\[
IR = \frac{\rho_b V_b}{\rho_s V_s}
\]

where \(\rho_b\) and \(\rho_s\) denote the mass densities of the backfill material and the soil, respectively, and \(V_b\) and \(V_s\) are the shear wave speeds of the two materials. Simply, IR can be used to determine whether the trench barrier is soft or hard, i.e., \(IR < 1\) means that the trench barrier is softer than the surrounding soil and visa versa.

5.1. Effects of geometrical parameters

In what follows, the mass density of the bentonite–soil mixture is assumed to be 1.2 t/m\(^3\), its shear wave speed is 30 m/s and its Poisson’s ratio is set to be 0.45. These values result in an impedance ratio of IR = 0.12, which indicates a very soft backfill material. The effects of the in-filled trench depth, width and distance on the reduction of the internal forces are shown in Figs. 8–10, respectively. As expected, the trends already observed in the case of an open trench also can be noted herein. Only some tolerances in the achieved reduction levels become obvious. Comparing the results shown in Figs. 2 and 8, it can be observed that the reduction values obtained using an in-filled trench is only about 70–80% of the corresponding reduction values obtained using an open trench. The difference between the reduction levels of the two types of trenches is reduced to be about 5–10% in the case of changing the trench width as could be seen from the comparison of Figs. 4 and 9. The most important difference is that there is no

![Fig. 8. Reduction of the internal forces due to different depths of the in-filled trench. (a) Reduction of the axial force, (b) reduction of the shearing force.](image-url)
amplification in the shearing force with the increase of the distance between the building and the in-filled trench as shown in Fig. 10. Instead, the reduction value tends to approach zero as the distance between the trench and the building increases.

The time history of the horizontal acceleration at point A, assuming \( L = 8 \text{ m} \), is depicted in Fig. 11. The results obtained in the case of no trench, an open trench, and an in-filled trench are compared. Different from the case of an open trench, the vibration waveform in the case of an in-filled trench is similar to the case of no trench with a certain time delay and reduced amplitudes. Thus, it may be stated here that the existence of the soft backfill material permits a considerable part of the waves to pass through the trench directly, although with reduced wave speed. The in-filled trench does not strongly enforce the body waves to travel vertically downwards into the soil. This behavior seems to produce a smaller amount of scattered waves than in the case of an open trench. Consequently, the generation
of new horizontal waves and a modification of its characteristics are also reduced. This could explain the time delay, the non-changed wave form observed in Fig. 11, and also the non-amplified shearing forces observed in Fig. 10. Moreover, this finding confirms the previous explanations of Figs. 6 and 7 in the case of an open trench.

5.2. Effects of the backfill material properties

In what follows, the analysis is performed for the case of an in-filled trench with a depth of 4.5 and a width of 0.5m, which is located at a distance of 2m from the building. The mass density of the backfill material is assumed to be 1.2t/m³, and its Poisson’s ratio is set to be 0.45. The shear wave speed of the halfspace soil is kept constant, while the corresponding wave speed of the backfill material is considered to be variable, taking the values given in Table 1. Consequently, the resulting impedance ratio IR varies between 0.12 and 0.36. Fig. 12 depicts the resulting reduction values versus the impedance ratio IR. With the increase of IR, the achieved reduction value decreases drastically, indicating that the softer trench is the more effective one. Noteworthy is the fact that as IR approaches zero, the achieved reduction values approach the values given in Fig. 2 for the open trench with depth of 4.5m. This can be easily conceived since the open trench is just a special case of in-filled trench with IR = 0.

The effects of changing the mass density of the backfill material $\rho_b$ as a ratio of the soil mass density $\rho_s$ are shown in Fig. 13(a). In fact, the mass density of the backfill can be adjusted by controlling the consistency of the bentonite-soil mixture, but the margin for varying the mass density is rather narrow in practice, therefore, the value of $\rho_b/\rho_s$ is allowed to range from 0.6 to 1. Fig. 13(b) shows the effect of changing the Poisson’s ratio on the force reduction level. From both figures, it can be concluded that the mass density as well as the Poisson’s ratio only have a very slight influence on the effectiveness of an in-filled trench.

6. Conclusions

A detailed investigation on the reduction of building vibrations due to a train passage using a trench barrier has been presented. A previously developed coupled BE–FE algorithm was applied for an analysis directly in the time domain. The effectiveness of using open or in-filled trench as a reduction measure has been demonstrated through a representative parametric study. Depending on the obtained results, the following conclusions may be drawn:
(1) Using open or in-filled trench barriers can reduce the vibrations of a structure and the resulting internal forces significantly. A reduction level up to 80% of the internal forces and vertical vibrations could be observed.

(2) The reduction in the column axial force is larger than the reduction in the shearing forces and bending moments.

(3) The use of an open trench is more effective than using an in-filled trench but its practical application is limited to relatively shallow depths. On the other hand, using softer backfill material increases the effectiveness of in-filled trench and allows for larger trench depth with no supporting measures of the vertical walls of the trench.

(4) Increasing the depth or the width of a trench leads to an increase in the obtained reduction level, but increasing the depth is by far more effective. Therefore, a feasibility study should be performed for each particular case to design the optimum trench with respect to its depth and width.

(5) Increasing the distance between the open trench and the building decreases (or even eliminates) its reduction effect due to the increase of the resulting scattered horizontal motions. This behavior is less pronounced in the case of an in-filled trench.

(6) The mass density and the Poison’s ratio of the backfill material have only slight effects on the isolation performance of the in-filled trench.

(7) The use of a raft foundation can reduce building response to train induced vibration significantly, but the additional cost should be taken into account. Moreover, the high reduction level of structure response associated with using trench barrier could not be achieved using raft foundation.

The current study aimed to provide a few general guidelines for the design of vibration isolation measures by means of trenches. The most important findings are summarized above. It should be noted, however, that in many practical cases it seems to be appropriate to perform a more detailed investigation of the structure/soil/trench system under consideration—similar as it has been done in this contribution. With the newly developed computer codes, where FEM and BEM are coupled directly in the time domain, these investigations can be done in a very efficient way.

Appendix A

A.1. Analytical and numerical approaches

A.1.1. Boundary integral formulation

The differential equation of Lamé–Navier describes the displacement field of a linear-elastic continuum which is subjected to dynamic loads. Under the assumption of homogeneous initial conditions and vanishing body forces this differential equation can be transformed to the boundary integral equation [32]

\[
c_{ik}u_{ik}(\xi, t) = \int \int u_{ik}^0(x, t, \xi, \tau) t_i(x, \tau) d\tau d\Gamma_x
\]

where \(u_{ik}^0\) and \(t_i\) are the fundamental solutions [33] for traction and displacement at the field point \(x\) at time \(t\) caused by a Dirac-load acting at the boundary point \(\xi\) at time \(\tau\). The fundamental solutions depend on the material parameters of the continuum, i.e., the velocities of the longitudinal and the transverse waves, \(c_p\) and \(c_s\), respectively and the mass density \(\rho\).

The \(u_i\) and the \(t_i\) represent the boundary values for the displacements and the traction, respectively. The integration is carried out over the boundary \(\Gamma\) with respect to \(x\) as indicated using \(\Gamma_x\). The matrix \(c_{ik}\) includes the integral-free terms, which depend on the geometry in the vicinity of the source point \(\xi\).

Generally, for each point on the boundary \(\Gamma\) either the displacements or the traction are known, and Eq. (A.1) is used to determine the unknown boundary values.

For the numerical solution, the boundary integral equation (A.1) is discretized in time and space and then solved. The time domain is subdivided into equal intervals where a constant, linear, or quadratic time interpolation function is used to represent the continuous time history of traction and displacements. Then, the time integration of Eq. (A.1) for the boundary values can be carried out analytically leading to functions that depend on space variables only. For an arbitrary boundary geometry, these functions cannot be integrated analytically. Therefore, the boundary \(\Gamma\) is divided into constant or quadratic isoparametric quadrilateral boundary elements with one or eight nodes per element, respectively. After discretization, Eq. (A.1) can be written in the form

\[
c_{ik}u_{ik}(\xi, t_N) + \sum_{j=1}^{L} \sum_{m=1}^{N} \int_{\Gamma_j} T_{ik}^{(N,m+1)}(x, \xi) d\Gamma_j u_{ik}^{(N,m)}
\]

where \(T_{ik}\) and \(U_{ik}\) are the traction and displacement kernels, respectively, resulting from the temporal integration of the fundamental solution. The outer summation in Eq. (A.2) is carried out over the total number of elements \(L\) and the inner one is carried out over the number of time steps \(N\). Gaussian quadrature is used to evaluate the integrals over the boundary \(\Gamma_x\). The quadrature scheme has to be modified for \(N = m\) to account for the singularities of \(T_{ik}\) and \(U_{ik}\) when the distance \(r\) between the source point \(\xi\) and the field point...
\( \mathbf{x} \) approaches zero. In this case, the traction kernel has a strong singularity (\( O(1/r^2) \)) and the corresponding integral exists in a principal value sense only. The displacement kernel has a weak singularity (\( O(1/r) \)), therefore its integral is regular.

After integration, Eq. (A.2) can be written in matrix notation as:

\[
\mathbf{U}^i N = \mathbf{T} u^N + \mathbf{E}^N,
\]

\[
\mathbf{E}^N = \sum_{m=1}^{N} \mathbf{T}_m u^{N-m+1} - \mathbf{U}_m N^{N-m+1}, \tag{A.3}
\]

where \( \mathbf{U}_m \) and \( \mathbf{T}_m \) are the coefficient matrices of the system at time \( m \Delta t \). For the current time step \( N \), all traction vectors \( \mathbf{T}_m \), \( m = 1–N \), and previous displacement vectors \( \mathbf{u}^N \), \( m = 1–N–1 \), are known.

\subsection*{A.1.2. Two-dimensional coupled BE–FE method}

In the following, a general two-dimensional BE–FE coupling formulation is presented where the problem in hand is divided into two parts as shown in Fig. 14.

The uniform half-space domain \( \Psi \) reduces to a plane region and is modeled by boundary elements. The compatibility and equilibrium conditions have to be satisfied along the common interface line between the two parts \( \Gamma_i \). It should be noted that the integral representation Eq. (A.1) holds in the same form for both, three-dimensional and two-dimensional problems. In the 2D case, however, it reduces to a line integral and the subscripts \( i, k \) take the values of 1 and 2 only.

The governing equation of motion of the finite element domain due to a time dependant applied load \( P(t) \) can be expressed and partitioned for the current time step \( N \) as:

\[
\begin{bmatrix}
\mathbf{M}_m & \mathbf{M}_m \backslash \\
\mathbf{M}_m & \mathbf{M}_m \backslash
\end{bmatrix}
\begin{bmatrix}
\dot{\mathbf{u}}_m \backslash \\
\dot{\mathbf{u}}_m \backslash
\end{bmatrix}
^N + 
\begin{bmatrix}
\mathbf{C}_m & \mathbf{C}_m \backslash \\
\mathbf{C}_m & \mathbf{C}_m \backslash
\end{bmatrix}
\begin{bmatrix}
\ddot{\mathbf{u}}_m \backslash \\
\ddot{\mathbf{u}}_m \backslash
\end{bmatrix}
^N
+ 
\begin{bmatrix}
\mathbf{K}_m & \mathbf{K}_m \backslash \\
\mathbf{K}_m & \mathbf{K}_m \backslash
\end{bmatrix}
\begin{bmatrix}
\mathbf{u}_m \backslash \\
\mathbf{u}_m \backslash
\end{bmatrix}
^N
= 
\begin{bmatrix}
\mathbf{P}_m \backslash \\
\mathbf{P}_m \backslash
\end{bmatrix}
^N,
\]

where \( \mathbf{M}, \mathbf{C}, \) and \( \mathbf{K} \) are the mass, damping and stiffness matrices, respectively. The vectors contain the nodal accelerations, velocities and displacements, respectively.

The variables on the interface \( \Gamma_i \) are denoted by the subscript \( i \) and the other variables are denoted by the subscript \( o \). Similarly, using the subscript \( i \) for the interface nodes and \( s \) for the surface nodes of the boundary element domain, Eq. (A.3) can be written as:

\[
\begin{bmatrix}
\mathbf{U}^i o \backslash \\
\mathbf{U}^i o \backslash
\end{bmatrix}
\begin{bmatrix}
\dot{\mathbf{u}}_o \backslash \\
\dot{\mathbf{u}}_o \backslash
\end{bmatrix}
^N + 
\begin{bmatrix}
\mathbf{T}_o \backslash \\
\mathbf{T}_o \backslash
\end{bmatrix}
\begin{bmatrix}
\ddot{\mathbf{u}}_o \backslash \\
\ddot{\mathbf{u}}_o \backslash
\end{bmatrix}
^N
+ 
\begin{bmatrix}
\mathbf{E}_o \backslash \\
\mathbf{E}_o \backslash
\end{bmatrix}
^N
= 
\begin{bmatrix}
\mathbf{P}_o \backslash \\
\mathbf{P}_o \backslash
\end{bmatrix}
^N.
\]

(A.5)

Employing the traction-free condition on the surface, and solving for the interface node variables, the nodal traction along the interface can be expressed as:

\[
\{t_i\}^N = [K_i] [u_i]^N - [E_{io}]^N,
\]

(A.6)

where \([K_i]\) and \([E_{io}]\) are the results from the necessary matrix and vector operations. The principle of virtual work is used employing interpolation functions over each boundary element to transform the traction in Eq. (A.6) into the nodal force vector \( \{P_{io}\} \):

\[
\{P_{io}\}^N = [K_{io}] [u_i]^N - [F_{io}]^N,
\]

(A.7)

where \([K_{io}]\) and \([F_{io}]\) represent \([K_i]\) and \([E_{io}]\) after transformation, respectively. Then, the principle of weighted residual is applied along the interface to obtain the coupled equation of motion as:

\[
\begin{bmatrix}
K_{io} & K_{oi} \\
K_{oi} & K_{io}
\end{bmatrix}
\begin{bmatrix}
\mathbf{u}_o \backslash \\
\mathbf{u}_i \backslash
\end{bmatrix}
^N = 
\begin{bmatrix}
\mathbf{P}_o \backslash \\
\mathbf{P}_i \backslash
\end{bmatrix}
^N.
\]

(A.8)

where the superscript * denotes the resulting coupled terms including the contributions of the boundary element domain as well as the mass and damping matrices of the finite element domain.

\subsection*{A.2. Verification}

To demonstrate the accuracy of the presented BE–FE coupling, it is applied to obtain the response of an elastic half-space under discontinues boundary stress distribution as shown in Fig. 15. This test example was used for comparison by many authors using different methods [34–37]. A vertical impulse traction \( q = 68.95 \text{ MPa} \) is applied on a strip of width \( b = 152.4 \text{ m} \). The elastic half-space has a Young's modulus \( E = 17,240 \text{ MPa} \),
mass density \( \rho = 3150 \text{kg/m}^3 \) and a Poisson’s ratio \( \nu = 0.25 \).

The vertical response is calculated at the points A, B, and C shown in Fig. 15. The results of the present method are given in Fig. 16(a) along with the results of another BEM three-dimensional approach [29]. The results of the other authors are scanned from reference [34] and displayed in Fig. 16(b). For the analysis using the three-dimensional boundary element approach, a length of about 3000 m is discretized in the \( x \)-direction and a width of about 1000 m in the \( y \)-direction. It can be observed from Fig. 16(a) that the results obtained by means of the three-dimensional computation are almost identical to those of the two-dimensional one. Moreover, all curves clearly demonstrate the good agreement of the results obtained by the presented methodology with those published by other authors. Comparing Fig. 16(a) and (b), it should be noted that the positive direction of the vertical displacements is defined differently.

Fig. 15. Half-space under surface vertical strip load impulse.

Fig. 16. Vertical responses calculated at points A, B, and C. (a) Results of the presented method (3D BE and 2D BE–FE [29]). (b) Results from other authors (after Abouseeda et al. [34]).

References


