Buildings in Barcelona Protected from Railway-Induced Vibrations with Base Isolation

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***Abstract*— This work describes two mid-height RC base isolated buildings recently erected in Barcelona. Both buildings are located near the main railway station in Barcelona (Barcelona-Sants), thus undergoing dynamic excitations due to intensive railway traffic. The base isolation system consists of spring-dashpot devices located at the ground level. The isolators were designed after the expected input according to extensive field measurements. The moderate levels of vibrations detected from numerical simulations (dynamic analyses) and from present observations confirm the efficiency of the base isolation systems.**

I. INTRODUCTION

Barcelona has a number of underground railway and metro (subway) lines, thus generating vibrations in nearby constructions. Recently, the construction of two buildings near the main railway station (Barcelona-Sants) was planned; both buildings will be located aside the wall of the railway tracks, therefore, important vibrations might be expected. Consequently, initial numerical studies were undertaken; the obtained results confirmed that in both buildings the expected levels of vibrations might be incompatible with the code requirements and the comfort of the occupants. The proposed solutions consisted of isolating the buildings with flexible supports; further numerical simulations showed significant reductions of the vibrations generated by railway traffic, and the proposed solutions were considered satisfactory. These solutions were implemented; the constructed building have exhibited adequate performance.

The first building is termed along this study "building A" and the second building is termed "building B". Both buildings have reinforced concrete structure; building A is 11-story and has housing use while building B is 5-story and has contains a hotel. Figure 1 display some views of building A.

(a) East façade (b) North façade (c) West façade

Figure 1. Building A

II. BUILDING A

A. General remarks

Building A was designed by architects M. Chalamanch, M. Lacasta and C. Santana (www.archikubik.com). As shown by Figure 1, building A is composed by a main body and a shorter body. The main body has eleven levels above the ground, being

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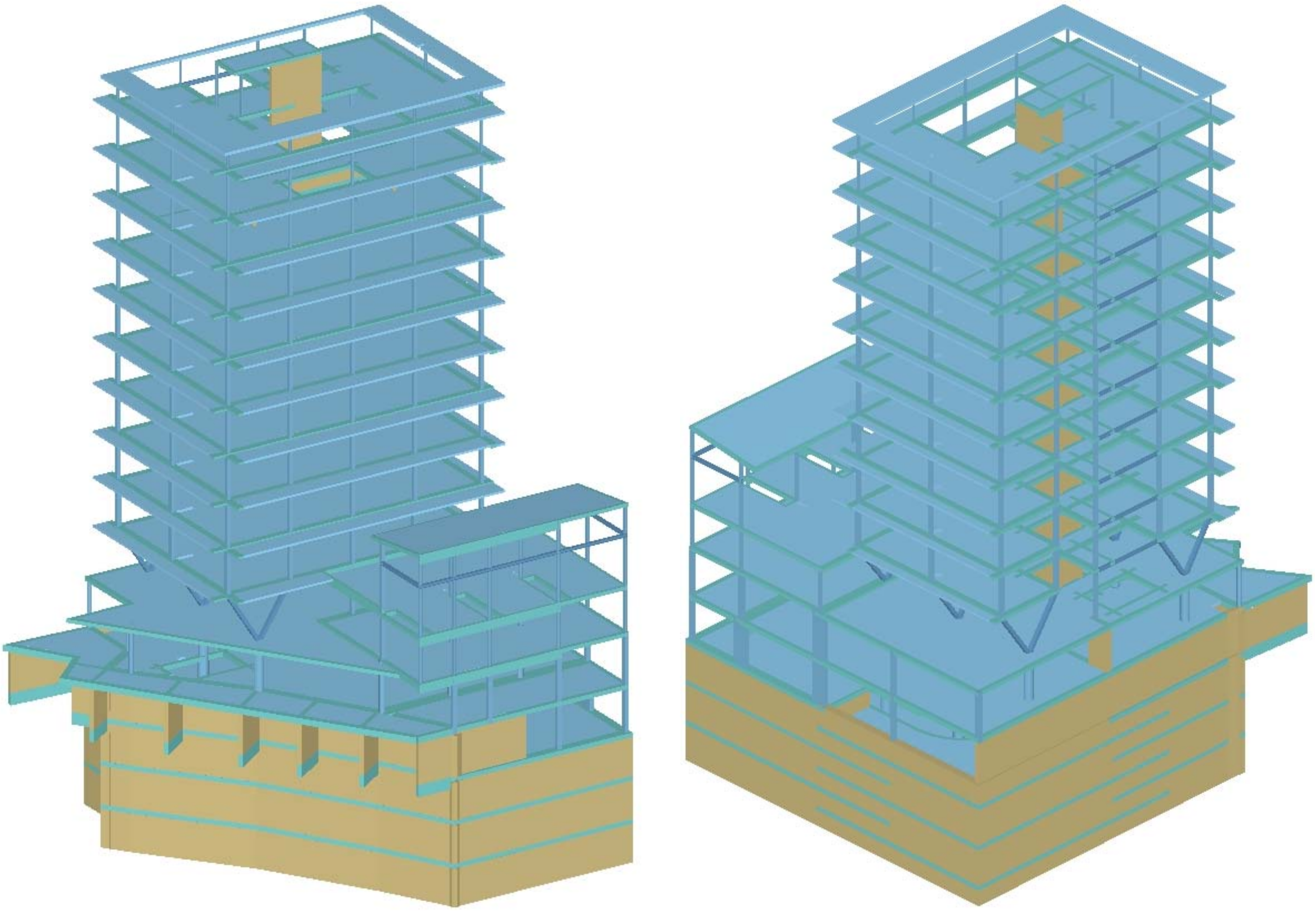
topped with a light roof; the smaller body has four floors. As well, there are four basement levels, while the top ones are not completely underneath the ground. The area of the basements levels is about 500 m^2, while the areas of the main body and of the shorter body are about 265 and 80 m^2, respectively. Figure 2 displays two views of the structure of building A. Remarkably, ninth story contains a swimming pool; the water volume is $10.45 \times 2 \times 1.50$ m^3. Another unique element are the V-shaped columns located on the first floor above the ground (see Figure 2); such columns allowed better design of the locations of the columns in the basements (for parking use) and in the floors (for commercial and housing use).

Figure 2 shows that the basement and the middle floor structure is formed by walls, columns and concrete slabs. The first floor has the same features and the upper floors have steel columns and concrete slabs. In some places, there are beams that protrude up to 15 cm below the slab. Given the poor soil characteristics, the building is supported by a deep foundation.



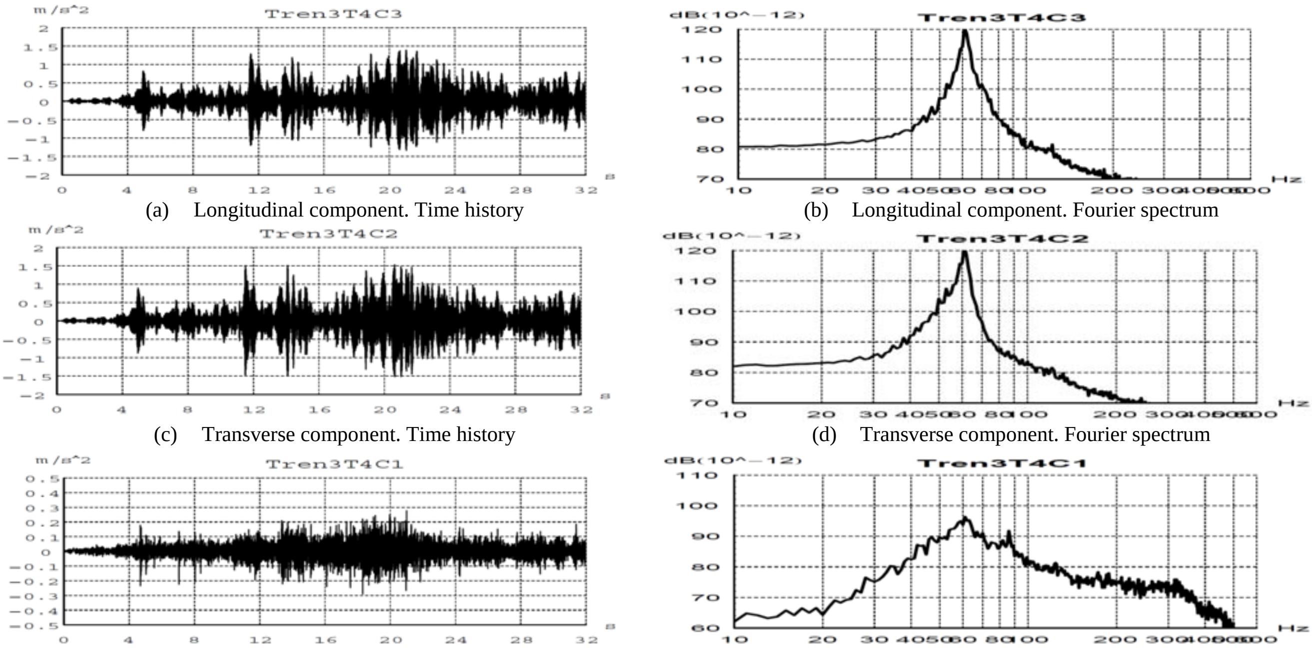
(a) Front view (wall aside the railway tracks) (b) Rear view

Figure 2. Structure of Building A

B. Field measurements

At the time the construction was planned, the closest track was about 20 m away from the wall which is aside the construction site. However, future plans involved to build new tracks in the available space; the closest distance from the building to the railway track will be only 5 m. These circumstances were taken into consideration in designing the field measurements; they consisted of recording accelerograms inside the railway station. Accelerometers were placed on the end portion of a platform situated between two tracks and on the wall adjacent the construction site; given that the closest track had not yet been built, the second type of registers were considerably lower. In both cases, accelerations were recorded in three directions: vertical and horizontal, parallel and transverse to the railway.

Figure 3 displays the vibrations recorded at the end of a platform and caused by the passing of two trains going in opposite directions. Figure 3 represents the time histories and the frequency content (Fourier transform) of the three components of the records.



(a) Longitudinal component. Time history (b) Longitudinal component. Fourier spectrum

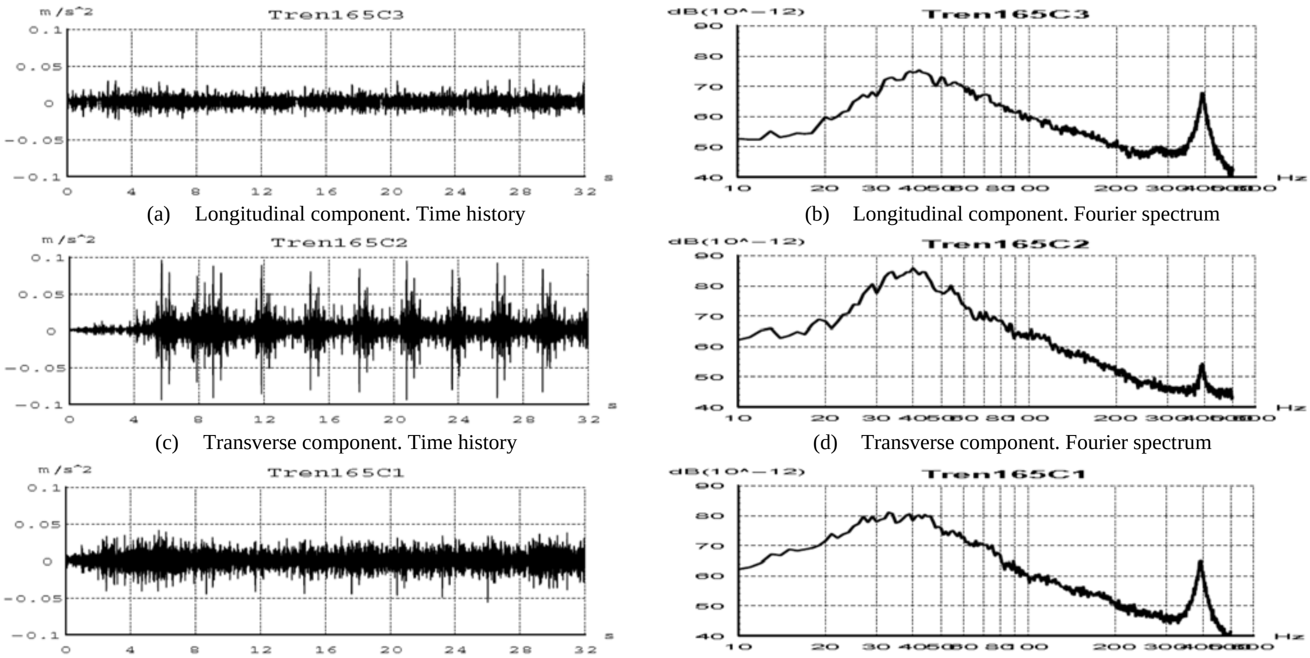
(c) Transverse component. Time history (d) Transverse component. Fourier spectrum

(e) Vertical component. Time history (f) Vertical component. Fourier spectrum

Figure 3. Registers in the platform between tracks 1 and 2. Building A

The spectra in Figure 3 show a significant peak (sharper in the horizontal components) located near 60 Hz. Since concentrated amplifications are not common in train-generated generated vibrations, it is suspected that it corresponds to a natural period of the platform. Since the peaks are more prominent in the horizontal direction, we can conclude that this mode of vibration is predominantly horizontal.

Figure 4 displays the vibrations recorded at the wall and caused by the passing of one train. Figure 4 represents the time histories and the frequency content (Fourier transform) of the three components of the records.



(a) Longitudinal component. Time history (b) Longitudinal component. Fourier spectrum

(c) Transverse component. Time history (d) Transverse component. Fourier spectrum

(e) Vertical component. Time history (f) Vertical component. Fourier spectrum

Figure 4. Registers in the wall adjacent to the construction site. Building A

The three signals in Figure 4 shows that there is some amplification in the direction perpendicular to the wall and that the motion in the longitudinal direction is particularly low. As well, comparison between Figure 3 and Figure 4 shows that the vibrations in the wall are significantly lower than those on the platform and that the 60 Hz peak is not present in the wall response. Given that the building will be erected behind the wall, the accelerograms displayed in Figure 4 are taken as design input.

C. Numerical modelling of the structural behavior

The linear structural behavior of the building has been described with a classical finite element mode implemented in the SAP software package [CSI 2008] version 8.1.2. The connections between the bottom basement walls and columns are modelled as clamped; the horizontal interaction between the soil and the basement walls is represented by horizontal springs; the assumed stiffness coefficient is 2/3 of the vertical (ballast) stiffness. The discretization of the structural members is described next.

- **Columns**. Columns (either reinforced concrete or steel) are described with 2-node 3D Euler-Bernouilli frame elements; each column is represented by a single element. In the concrete columns, the stiffness corresponds to the gross section, neglecting the reinforcement contribution.
- **Slabs**. Slabs are described with 4-node 2D Kirchhoff shell elements. Cracking is taken into account by reducing up to 85% the bending stiffness.
- **Beams**. Beams are described with 2-node 3D Euler-Bernouilli frame elements; each beam is discretized such as the modes coincide with those of the adjoining slabs. Cracking is taken into account by reducing up to 85% the bending stiffness.

Walls. Walls are described with 4-node 2D Kirchhoff shell elements.

Damping is described by a viscous model, damping factor is 1%.

Noticeably, the cooperation of the non-structural elements (e.g. cladding, partitioning and infill walls, stairs, and staircases) has been neglected. Figure 5 displays the finite element mesh together with the axis considered in the analysis; *x*/*y* directions are perpendicular/parallel to the railway track.

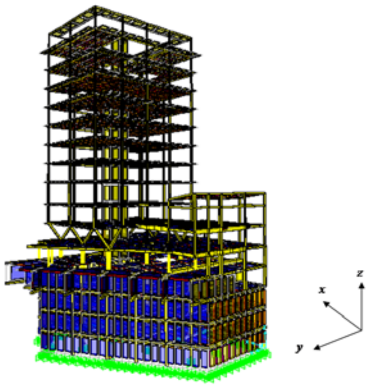


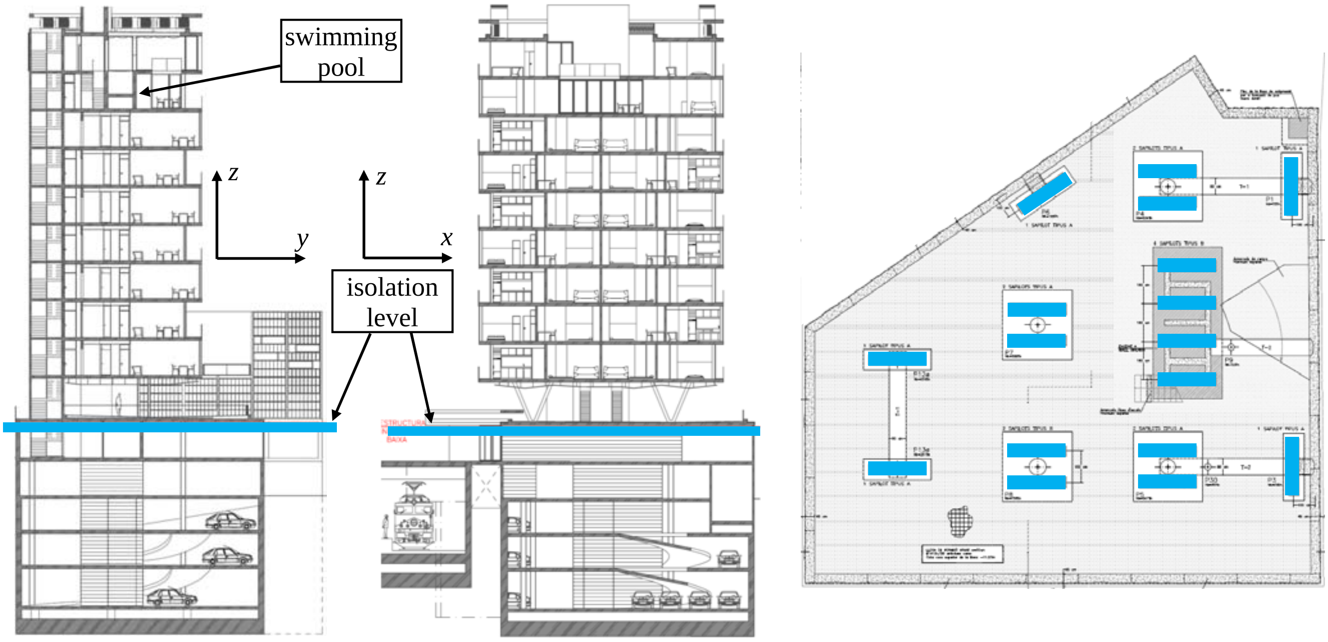
Figure 5. Finite element mesh for Building A

Two types of analyses have been carried out with the described model:

- **Modal analysis**. The modal characteristics of the building have been obtained by classical linear eigenvalue analysis.
- **Dynamic analysis**. This analysis provides the time history response to the expected input accelerograms displayed in Figure 4. A damping factor equal to 1% is assumed. In the time integration, Δt = 0.001 s.

D. Proposed solution

Since initial numerical simulations showed vibration amplitudes highly above the thresholds indicated by the considered codes [ISO 2631-1 1997, ISO 2631-2 2003], the use of base isolation was suggested. Several types of isolators were contemplated; those consisting in helical steel springs and hydraulic dampers were deemed more suitable. Figure 6 displays two elevation views and a plan view of the building indicating the suggested position of the isolators; that location was chosen because the floors which are under this level do not require a high degree of isolation, given its parking and commercial use. As well, the layout under the V-shaped columns was easier since a smaller number of devices was required (Figure 6.c).



(a) Longitudinal section (b) Transverse section (c) Plan view
Figure 6. Proposed location of the isolators. Building A

The design of the isolators was carried out iteratively on a trial-and-error procedure accounting for the axial forces in the devices and the dynamic response of the isolated building. As in building B, the characteristics of the proposed devices are: vertical stiffness 0.41 kN/mm; horizontal stiffness 0.31 kN/mm and damping factor approximately 0.04. A simplified modal analysis showed that the vertical frequency is about 3.22 Hz.

E. Numerical results of the modal analysis

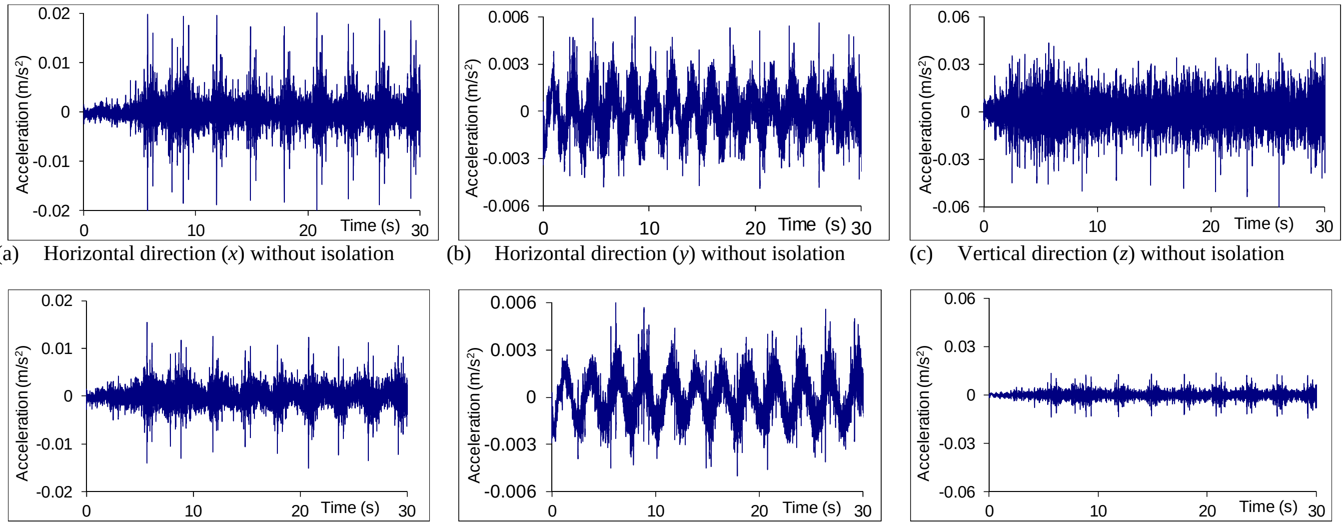
As discussed previously, the modal characteristics of the building have been obtained by classical linear eigenvalue analysis. TABLE 1 displays the periods and the shapes of the first six modes. TABLE 1 shows that the modes corresponding to vertical vibration of the walls have significantly shorter periods. Comparison among the values with and without isolators shows that those devices significantly elongate the periods, i.e. introduce flexibility in the building; this effect is more intense for translational modes than for torsional ones.

TABLE 1. Modal parameters of building A

	Without isolators		**With isolators**	
Mode No.	**Period (s)**	**Shape**	**Period (s)**	**Shape**
1	2.64	Torsion	3.21	Translation *xz*
2	2.50	Translation *xz*	2.75	Torsion
3	1.88	Translation *yz*	2.56	Translation *yz*
4	0.93	Torsion (second mode)	1.08	Torsion (second mode)
5	0.78	Torsion (third mode)	0.82	Torsion (third mode)
6	0.69	Translation *xz* (second mode)	0.77	Translation *xz* (second mode)

F. Numerical results of the dynamic analysis

This analysis provides the response to the simultaneous actuation of the three components of the selected input depicted in Figure 4. Figure 7 represents three plots of the time-history accelerations in horizontal and vertical directions in a characteristic point of the 10th story slab; this point has been chosen because the deflections were higher than in the other considered positions. Results in the other floor slabs situated above the isolation level were similar. Comparison among the values with and without isolators shows that those devices significantly reduce the acceleration amplitudes, mainly in the vertical direction; noticeably, this reduction is most needed, since vertical accelerations are much higher than the horizontal ones. The reduction in the horizontal *y* direction (parallel to the railway track) is practically inexistent; this circumstance is irrelevant, given the low vibration levels in this direction.



(a) Horizontal direction (*x*) without isolation (b) Horizontal direction (*y*) without isolation (c) Vertical direction (*z*) without isolation

(d) Horizontal direction (*x*) with isolation (e) Horizontal direction (*y*) with isolation (f) Vertical direction (*z*) with isolation

Figure 7. Time-history acceleration responses in the 10th floor. Building A

The comparison between the obtained acceleration amplitudes and the threshold levels indicated in the codes [ISO 2631-1 1997, ISO 2631-2 2003] is established in terms of the weighted mean quadratic acceleration a_v given by $a_v = \sqrt{(k_x\, a_{wx})^2 + (k_y\, a_{wy})^2 + (k_z\, a_{wz})^2}$, where a_{wx}, a_{wy} and a_{wz} are the weighted r.m.s. (root mean square) accelerations in directions *x*, *y* and *z*, respectively; the values of the weighting factors are $k_x = k_y = 1.4$ and $k_z = 1$. The weighted r.m.s. accelerations are determined as $a_w = \sqrt{\sum_i (W_i\, a_i)^2}$ where W_i are the weighting factors for the *i*-th one-third of octave band. TABLE 2 displays the weighted mean quadratic accelerations a_{wx}, a_{wy}, a_{wz} and a_v in all the slabs of building A. Figures in TABLE 2 confirm basically the conclusions derived after Figure 7; reductions below the isolation level (– 4 to – 1, indicated with grey in TABLE 2) are neglectable while accelerations above that level (1 to roof) are much higher. The average reduction in the vertical direction is 81,50% while a_v is reduced about 78%.

TABLE 2. Weighted mean quadratic acceleration in building A without / with isolators

Level	**a_{wz} (m/s²)**	**a_{wx} (m/s²)**	**a_{wy} (m/s²)**	**a_v (m/s²)**
– 4	0.0671 / 0.0653	0.0116 / 0.0116	0.0088 / 0.0088	**0.0701** / **0.0684**
– 3	0.0671 / 0.0638	0.0115 / 0.0115	0.0088 / 0.0088	**0.0701** / **0.0670**
– 2	0.0670 / 0.0620	0.0114 / 0.0114	0.0087 / 0.0088	**0.0699** / **0.0652**
– 1	0.0671 / 0.0595	0.0108 / 0.0109	0.0081 / 0.0084	**0.0697** / **0.0625**
1	0.0680 / 0.0174	0.0084 / 0.0072	0.0059 / 0.0041	**0.0695** / **0.0209**
2	0.0673 / 0.0140	0.0051 / 0.0050	0. 0051 / 0. 0036	**0.0681** / **0.0164**
3	0.0685 / 0.0166	0.0042 / 0.0044	0.0053 / 0.0034	**0.0691** / **0.0183**
4	0.0684 / 0.0142	0.0051 / 0.0046	0.0058 / 0.0035	**0.0692** / **0.0163**
5	0.0683 / 0.0111	0.0053 / 0.0046	0.0059 / 0.0034	**0.0692** / **0.0137**
6	0.0679 / 0.0085	0.0045 / 0.0041	0.0053 / 0.0030	**0.0686** / **0.0111**
7	0.0673 / 0.0068	0.0037 / 0.0036	0.0040 / 0.0022	**0.0677** / **0.0090**
8	0.0678 / 0.0060	0.0017 / 0.0017	0.0025 / 0.0016	**0.0680** / **0.0068**
9	0.0584 / 0.0152	0.0024 / 0.0022	0.0030 / 0.0020	**0.0587** / **0.0158**
10	0.0720 / 0.0157	0.0051 / 0.0051	0.0050 / 0.0031	**0.0727** / **0.0178**
Roof	0.0717 / 0.0125	0.0082 / 0.0091	0.0076 / 0.0043	**0.0733** / **0.0189**

G. Final considerations

As discussed previously, the construction of the building has been completed, without major problems; noticeably, the observed levels of vibration are acceptable. Figure 8 displays some views of the building with the isolators.

(a) Location of isolators on the top of a column (enlarged column head)

(b) Location of isolators on the stairs and elevators case

(c) Global view

Figure 8. Images of Building A with isolators

III. Building B

A. Description of the building

The building has two basements and five stories, and has a framed reinforced concrete structure with rectangular columns and constant-depth slabs.

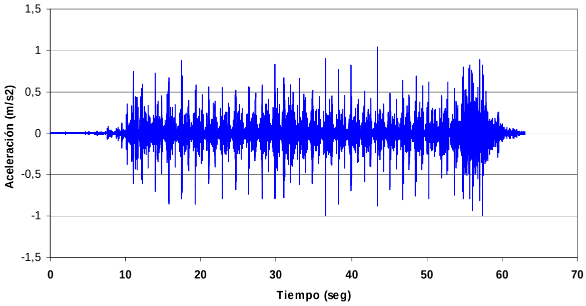
B. Field measurements

Field measurements consisted of:

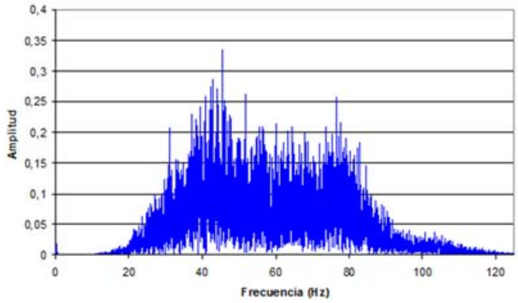
- Acceleration recordings at the construction site (building lot). Records were taken in five points; one of them was located aside the wall nearest the railway trucks. Accelerations were recorded in three directions: vertical and horizontal, parallel and transverse to the railway.
- Acceleration recordings inside the railway station. Accelerometers were placed on the ground, at the railway level; sensors on platforms or other structural elements have been avoided because of the possible alterations of the characteristics of the signal. Vibrations caused by the passing of two trains were registered; in the first train, the distance to the sensor was 5.4 m, and in the second train, the distance was 1.7 m.

In all the cases, each register lasted at least 60 s; this duration was considered enough to characterize the accelerograms.

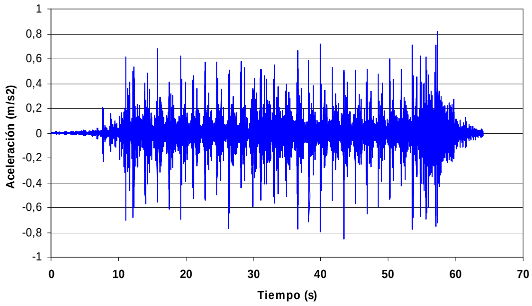
The closest distance from the building to the railway track will be 5 m; however, in the current situation, the closest track in use is located approximately 20 m away. Therefore, to select the design input, two approaches were considered: (i) to use empirical attenuation formulae to convert the recordings in the construction site into equivalent input corresponding to 5 m distance, and (ii) to use directly the recordings inside the railway station. With the first approach using the well-known Bornitz formula [Bornitz 1931], the obtained input accelerations were clearly underestimated for frequencies higher than 20 Hz; for this reason, the second approach was chosen, and the record which was taken 5.4 m away from the passing train is selected as the design input. Figure 9 represents the time histories and the frequency content (Fourier transform) of the three components of the selected input.



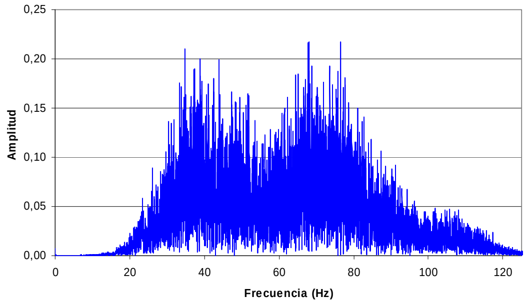
(d) Longitudinal component. Time history



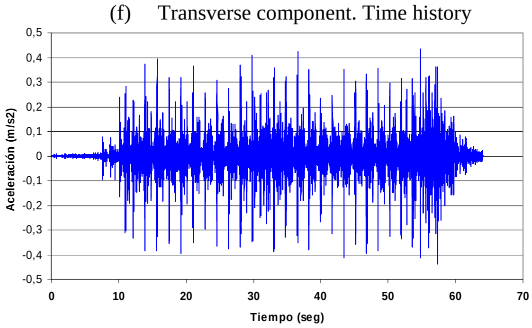
(e) Longitudinal component. Fourier spectrum



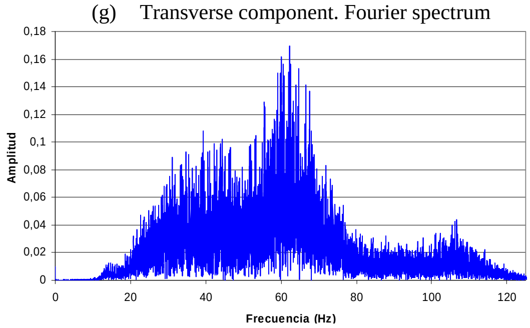
(f) Transverse component. Time history



(g) Transverse component. Fourier spectrum



(h) Vertical component. Time history

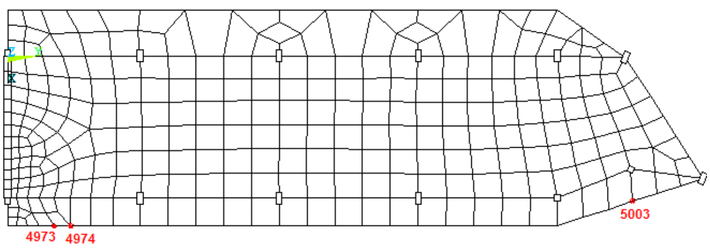


(i) Vertical component. Fourier spectrum

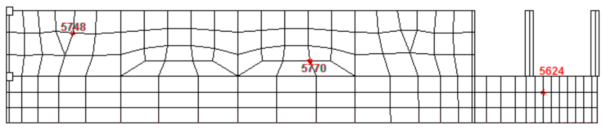
Figure 9. Design input (5.4 m distance) for Building B

C. *Numerical modelling of the structural behavior*

The linear structural behavior of the building has been described with a classical finite element mode implemented in the ANSYS software package [ANSYS 2005]. Columns are discretized with 2-node frame elements (at least four elements are used per column) and basement walls and slabs are discretized with 4-node shell elements. The concrete deformation modulus is selected as suggested by the Spanish design code [EHE-98 1998] for instantaneous loads; given that the characteristic value of the concrete compressive strength is f_{ck} = 30 MPa, the obtained modulus is E = 10000 $(f_{ck} + 8)^{1/3}$ = 33600 MPa. Since the considered situations correspond to serviceability conditions, the assumed loading combination is $D + L$ (dead + live loads), without further safety factors. Noticeably, the cooperation of the non-structural elements (e.g. cladding, partitioning and infill walls, stairs, and staircases) has been neglected. Figure 10 displays representative views of the finite element mesh.



(a) Basement slab



(b) Basement wall

Figure 10. Finite element mesh for Building B

Three types of analyses have been carried out with the described model:

- **Modal analysis**. The modal characteristics of the building have been obtained by classical linear eigenvalue analysis.

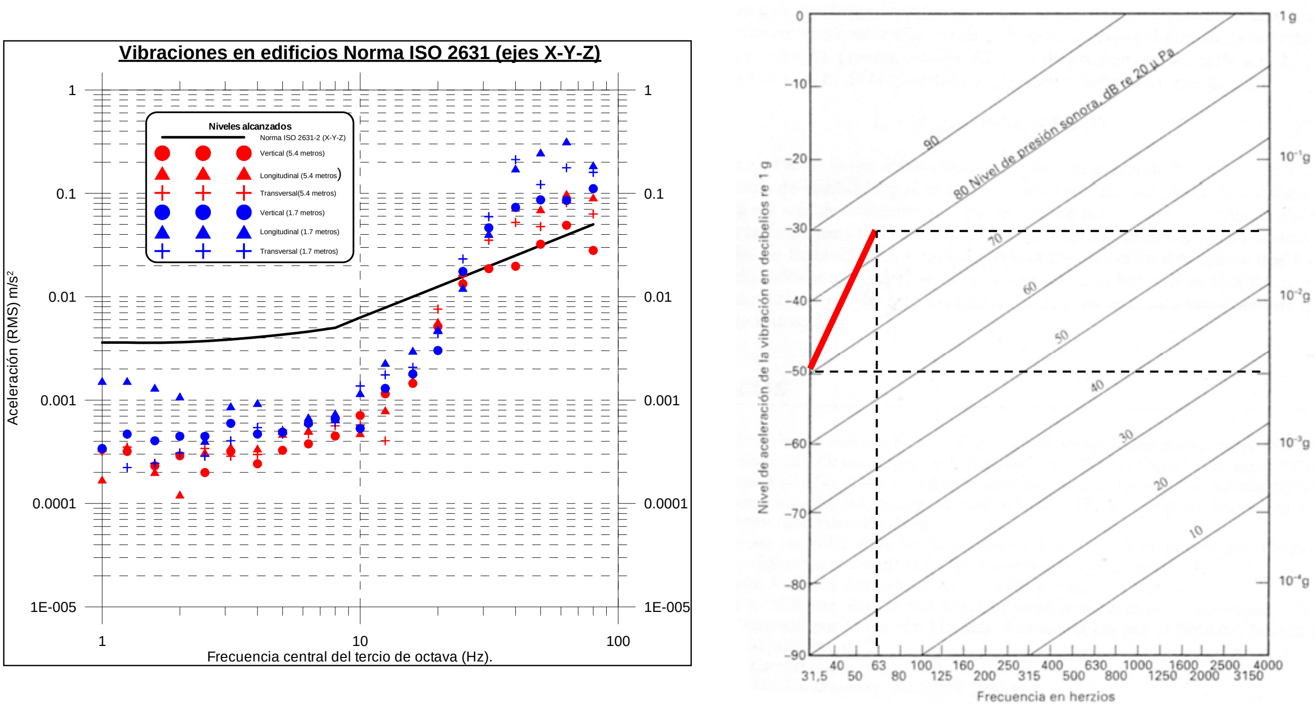
- **Harmonic analysis**. This analysis provides a scanning of the structural response to harmonic waves. The considered input is the simultaneous actuation of the three components of acceleration (longitudinal, transverse and vertical). The output is the dynamic amplification vs. the input frequency.
- **Spectral analysis**. A smoothed response spectrum has been derived for the design accelerogram (Figure 9); then the responses of the first 100 modes are obtained and they are combined by a “complete quadratic combination” (CQC) criterion [Wilson et al. 1979].

D. Proposed solutions

To assess the risk of excessive vibrations generated by railway traffic, the results of the numerical analyses have been compared with the bounds indicated in the design codes. Two types of comparisons have been carried out: the vibrations of the slabs and the noise emission.

Vibrations of the slabs. The reference code is [ISO 2631-2 2003]. Figure 11.a shows the comparison among the limit acceleration and the obtained accelerations. In Figure 11.a, red and blue points correspond to the source of vibrations 5.4 and 1.7 m away, respectively. Figure 11.a shows that, for frequencies higher than about 12 Hz, the obtained amplitudes clearly exceed the reference values.

Noise emission. The reference text is [Harris 1997] and the reference code is [NBE-CA-88 1988]; Figure 11.b displays a chart providing the expected noise intensity (dB) corresponding to the frequency and acceleration of the emitting panel (walls or slabs). Figure 11.b shows that the expected noise level for the considered rank of frequencies is about 75 dB; this intensity is considered excessive [NBE-CA-88 1988].



(a) Vibration of the slabs (b) Noise

Figure 11. Comparison between the obtained amplitudes and the design constraints. Building B without isolation

Since the structural vibrations are excessive, some solutions are suggested:

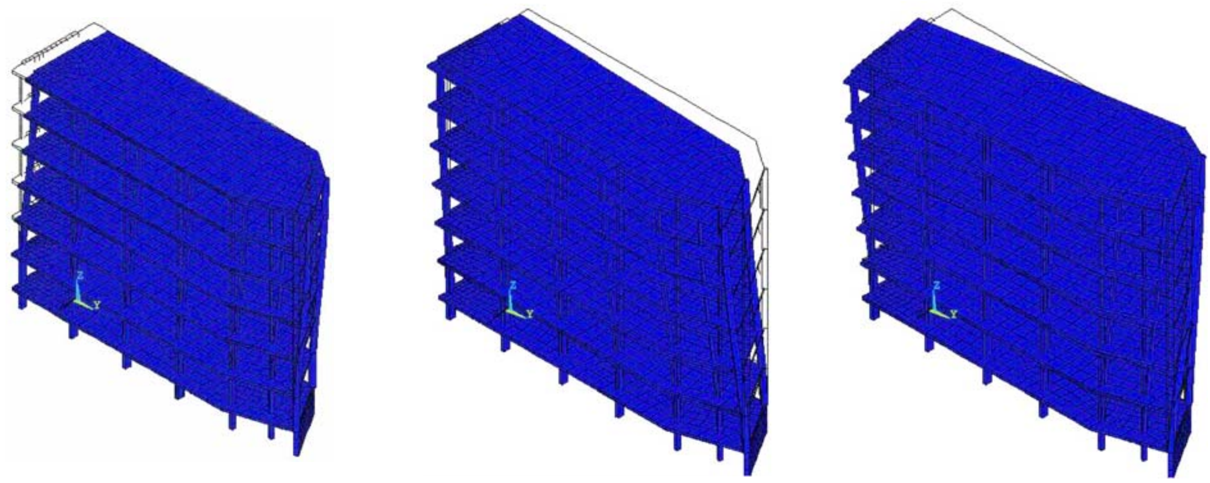
- **Steel spring isolators**. These elements consists of conventional helical steel springs parallel connected with hydraulic (oil) dampers. Their main practical application is building isolation against vibrations caused by railway traffic. Their natural frequencies range in between 2.5 and 5 Hz. They are very durable and resistant to fatigue and fire. In general, these devices do not compromise the wind strength of the building. In the building under consideration, it is recommended to place this type of elastic supports under the ground floor slab; this arrangement limits the vibration reduction to the above floors (where it is most needed), while reduces the weight to be resisted. The supports should be placed on top of the basement columns.
- **Rubber isolators**. Reinforced elastomeric bearings specifically designed for vibration control [Castellano et al. 2006] are considerably cheaper than metal springs; however, present major problems with respect to durability, fire resistance and fatigue. As well, their low frequency vibration behavior should be checked, because there is some risk of coupling with their lower frequency modes.

- **Rubber sheets**. Some buildings have been isolated by continuous elastomeric sheets (layers) being placed at the foundation level. This solution can be effectively and economically advantageous to both vibration and noise reductions. However, the fact that the building has deep foundation, poses certain difficulties. Moreover, replacement in case of aging or other deteriorations is almost impossible.

According to these considerations, the use of steel springs as bearing isolators is proposed. The characteristics of the proposed spring are: vertical stiffness 0.41 kN/mm; horizontal stiffness 0.31 kN/mm; damping: approx. 4% of the critical value; vertical frequency: approx. 3.5 Hz. After modifying the structural model of the building with the inclusion of the springs, new structural analyses (modal, harmonic and spectral) have been carried out. These analyses have confirmed the high effectiveness of the springs to reduce the accelerations and displacements; specifically, the maximum accelerations experienced a reduction of two or three orders of magnitude. The building satisfactorily meets the criteria of ISO 2631. The natural vertical frequency of the spring does not have relevant influence since it is in a range for which the building does not experience dynamic amplification effects.

E. Numerical results of the modal analysis

As discussed previously, the modal characteristics of the building have been obtained by classical linear eigenvalue analysis. Figure 12 displays the shapes of the first three modes, together with the corresponding natural frequencies. Figure 12.a, Figure 12.b and Figure 12.c show that the first, second and third modes correspond principally to translation in the long direction, to translation in the short direction and to torsion, respectively. Comparison between the frequencies without and with isolation confirms the significant increase of flexibility generated by the isolators.



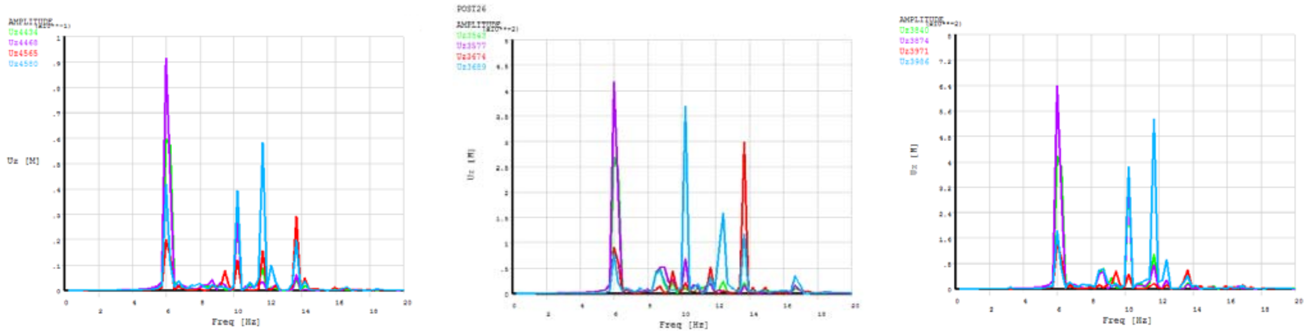
(a) First mode (0.514 / 0.199 Hz) (b) Second mode (0.581 / 0.205 Hz) (c) Third mode (0.661 / 0.319 Hz)

Figure 12. Modal shapes of Building B without / with isolators

F. Numerical results of the harmonic analysis

This analysis provides the response to the simultaneous actuation of the three components of harmonic waves (longitudinal, transverse and vertical). The acceleration amplitudes are chosen according to those of the selected input depicted in Figure 9. The chosen amplitudes are: longitudinal: 1.036 m/s^2, transverse: 0.85 m/s^2, and vertical: 0.437 m/s^2.

Figure 13 represents three spectra describing the amplifications of the vertical displacement (deflection) in a number of points of the building slabs (without isolation) under constant-amplitude harmonic waves. The frequency rank is 0-20 Hz.



(g) Fourth floor slab (h) First floor (i) Second floor

Figure 13. Amplifications of the vertical displacement in a number of points of Building B (without isolation) under constant-amplitude harmonic waves

Comparison between the spectra in Figure 13 and Figure 9 show that the peaks in Figure 13 correspond to natural frequencies of the slabs rather than to input peaks. Comparison with Figure 12 shows that the first three modes have significantly lower frequencies; in fact, the first major peak (about 6 Hz) corresponds to the 14^{th} mode.

G. Numerical results of the spectral analysis

Inside the spectral analysis approach, Figure 14 displays the design spectra for the longitudinal, transverse and vertical directions, respectively.

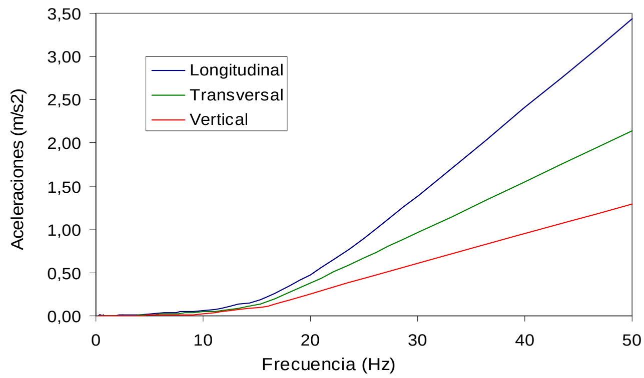


Figure 14. Design spectra for the longitudinal, transverse and vertical directions (Building B)

The spectra in Figure 14 have been obtained with SeismoSignal software code [SeismoSoft 2014].

TABLE 3 presents the values of the maximum response acceleration obtained after the design spectra depicted in Figure 14. Codes in brackets indicate the location of the points; BW: Basement Wall, BS: Basement Slab, RS: Roof Slab, and 1S: 1^{st} floor Slab.

TABLE 3. Maximum output acceleration (m/s^2) for building B without / with isolators

Output acceleration direction	Input spectrum		
	Transverse	Longitudinal	Vertical
Transverse	0.148 / 0.000934 [BW]	1.8451 / 0.00283 [RS]	0.370 / 0.000249 [BW]
Longitudinal	0.127 / 0.0000589 [BS]	1.849 / 0.00761 [RS]	0.123 / 0.000209 [BW]
Vertical	0.171 / 0.000240 [BS]	1.676 / 0.00372 [1S]	0.1283 / 0.000640 [BS]

Comparison between the results of the harmonic and spectral analyses show that the agreement is satisfactory; the amplitudes from the spectral analysis are lower than those of the harmonic analysis. Comparison between the results without and with isolators show the important reduction of the acceleration.

IV. CONCLUSIONS

Two mid-height RC buildings have been recently designed and erected in Barcelona; design involved base isolation to mitigate railway traffic-generated vibrations. The base isolation system consists of helical steel springs and dashpot devices located at the ground level. The design has consisted of measuring the expected input and to performing linear dynamic analyses to evaluate the vibration levels. The proposed solutions provides moderate levels of vibrations; this is detected from numerical simulations and from present observations.

REFERENCES

[1] ACHE (Asociación Científico-técnica del Hormigón Estructural). Problemas de vibraciones en estructuras (E-8). Colegio de Ingenieros de Caminos, Canales y Puertos (1991).

[2] Alarcón G., Solé J., Romeu J., Cardona J., Test and prediction of free field vibrations caused by high-speed trains near the Spanish candidate ITER site, Proc. 11th International Congress on Sound and Vibration, St. Petersburg, Russia, 2004.

[3] Amick H., A frequency-dependent soil propagation model, SPIE Conference on Current Developments in Vibration Control for Optomechanical Systems, Denver, Colorado, 1999.

[4] Amick H., Gendreau M. Construction vibrations and their impact on vibration-sensitive facilities. ASCE Construction Congress, Orlando. EEUU, 2000.

[5] ANSYS® Academic Research, Release 15.0. 2005.

[6] ATC (Applied Technology Council). ATC Design Guide 1. Minimizing Floor Vibration. ATC (1999).

[7] Bachmann H., Ammann W. Vibrations in Structures Induced by Man and Machines. IABSE (1987).

[8] Bornitz, G. (1931) Uber die Ausbreitung der von Groszkolbenmaschinen erzeugten Bodenschwingungen in die Tiefe, J. Springer (Berlin).

[9] Car E., López Almansa F., Oller. S. Estructuras sometidas a acciones dinámicas. CIMNE (2000).

[10] Castellano M.G., De Toni S., Testa M., Tisalvi M. (2006) Elastomeric Vibration Isolators for Railway Lines. Proceedings of the 6th World Congress on Joints, Bearing, and Seismic Systems for Concrete Structures, Halifax, Nova. Scotia, Canada.

[11] Crone A., Astrup T. and Finne P. Prediction of vibrations and structure-borne noise from railways. Inter-Noise 99, Fort Lauderdale, Florida, EEUU, 1999.

[12] CSI Anal y sis Reference Manual for SAP2000®, ETABS®, and SAFE™. 2008. Computers and Structures, Inc.

[13] Degrande G., Schillemans L. Free field vibrations during the passage of a Thalys highspeed train at variable speed. Journal of Sound and Vibration, 247. 2001.

[14] DIN 4150. Vibrations in buildings. Part 1: Prediction of vibration parameters. Part 2: Human exposure to vibrations in buildings. Part 3: Effects of vibrations on structures. 1999.

[15] Dowding C.H., Construction vibration, Prentice-Hall, 1996.

[16] EHE-98. Instrucción de Hormigón Estructural. Comisión Permanente del Hormigón. Ministerio de Fomento. 1998.

[17] EEA Technical report No 11/2010. Good practice guide on noise exposure and potential health effects. European Environment Agency.

[18] Gutowski T.G., Dym C.L., Propagation of ground vibration: a review, Journal of Sound and Vibration 49(2), 179-193, 1976.

[19] Harris C.M. Manual de Medidas Acústicas y de Control del Ruido. McGraw-Hill 1998.

[20] IS0 2631-1. (1997). Mechanical vibration and shock- Evaluation of human exposure to whole-body vibration - Part 1: General requirements. International Organization for Standardization.

[21] IS0 2631-2. (2003). Mechanical vibration and shock- Evaluation of human exposure to whole-body vibration - Part 2: Vibration in buildings (1 Hz to 80 Hz). International Organization for Standardization.

[22] Kim D., Lee J., Propagation and attenuation characteristics of various ground vibrations, Soil Dynamics and Earthquake Engineering 19, 115-126, 2000.

[23] Krylov V.V. Ground vibrations generated by superfast trains. Noise & Vibration worldwide, vol.25, No. 6. 1994.

[24] Llei de protecció contra la contaminació acústica. Generalitat de Catalunya, Departament de Medi ambient, 2002.

[25] Luna R., Jadi H. Determination of dynamic soil properties using geophysical methods. Proceedings of the First International Conference on the Application of Geophysical and NDT Methodologies to Transportation Facilities and Infrastructure, S. Louis. EEUU, 2000.

[26] Mead D.J. Passive Vibration Control. John Wiley (1998).

[27] NBE-CA-88. Condicionas acústicas en los edificios. Ministerio de Fomento, 1988.

[28] Rossi F. Soil vibration by high-speed trains: an evaluation method. Transport Noise and Vibration, St. Petersburg. Russia, 2002.

[29] SeismoSoft. SeismoSignal code is downlodable at http://www.seismosoft.com/en/SeismoSignal.aspx. 2014.

[30] Wilson E.L., Der Khiuregian A., Bayo E.P. A Replacement for the SRSS Method in Seismic Analysis, Earthquake Engineering & Structural Dynamics Vol. 9 187-192 (1981).