

UNIVERSIDAD POLITÉCNICA DE MADRID
Escuela Técnica Superior de Ingenieros de Caminos, Canales y
Puertos



**Constrained layer damping applied to
lightweight composite floors: modelling,
testing and structural optimization**

DOCTORAL THESIS

Submitted for the degree of Doctor by:

Carlos Martín de la Concha Renedo
Civil engineer

Madrid, 2023



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Under the supervision of:
Dr. Ivan Muñoz Díaz
Dr. Jaime Higinio García-Palacios

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Abstract

The present doctoral thesis addresses an increasingly concerning problem when designing modern buildings: floors' vibration issues due to human induced excitation. Nowadays, whenever a floor design is susceptible to exceed the Vibration Serviceability Limit State (VSLS), something common in composite or timber construction, design engineers tend to oversize the structure by increasing its mass or its stiffness. This solution is not the most efficient and indeed, degrades the green credentials that modern building floors should try to achieve by means of reducing as much as possible their embodied Carbon (EC). This thesis claims that there exist an alternative for designing serviceable floors while keeping their lightweight and slender nature. This solution consists on increasing the floor's damping since the design stage. Therefore, floors could be designed considering the integration of damping devices that enhance their dynamic performance, while the mass and the stiffness are used to comply with static limit states.

The research conducted has investigated an specific damping technology applicable to lightweight composite floors which is called Constrained layer damping or CLD. More specifically, the treatment studied was developed by ARUP at the beginning of the 2000's under the name 'resotec' and has demonstrated to be efficient enough in reducing the floor resonant responses in office buildings. This treatment consist on a thin Viscoelastic (VE) layer located between the concrete slab and the steel profile of a floor's composite beam. When the this one vibrates in bending, the VE is deformed to shear dissipating additional energy, and thus, increasing the floor damping ratio. This treatment has been usually applied along a proportion of the beam length near the supports, leaving the central region of the beam connected to longitudinal shear by means of studs, in order to not loose so much bending stiffness. In consequence, this solution implies designing a floor in which composite effect is not fully exploited, but with a better dynamic performance. Despite this technology has already been applied in real building project demonstrating its efficacy, its whole potential has not been fully investigated. In addition, although use full, this technology has not been spread as initially expected. This is understandable, as current engineers do not have the enough perspective, knowledge or tools to know when this solution might be effective, and if yes, how to accurately design it or even

predict the additional damping ratio it could provide. This thesis aims to fill this gap by realizing a holistic study of CLD treatments applied to composite floors. For that, the modelling of these treatments is firstly studied in detail. A deep comparison has been performed between different modelling approaches from analytical Partial Differential Equations (PDEs) solved by sections (to reproduce partially-treated beams) to detailed FE models. Modal analysis has been the main tool used for this. Then three reduced scale models of CLD-treated composite beams have been tested to proof the effectiveness of this damping treatment and the accuracy of the models on assessing it. Finally, an application example in which different composite floors have been design with and without considering the use of this technology has been carried out. For that, a multi-objective structural optimization problem has been proposed. This whole research not only provides engineers tools to design this treatments and predict their behaviour, but also proofs that in certain cases, their use leads to more lightweight, comfortable and sustainable floors.

Resumen

La presente tesis doctoral aborda un problema cada vez más preocupante en el diseño de edificios modernos: las vibración excesiva de los forjados debida a excitaciones inducidas por humanos. En la actualidad, cuando el diseño de un forjado es susceptible de no cumplir con el Estado Límite de Servicio de Vibraciones (ELSV), algo que ocurre con frecuencia al diseñar forjados mixtos o de madera, el calculista tiende a sobredimensionar la estructura aumentando su masa o su rigidez. Esta solución no es la más eficiente y, de hecho, degrada el caracter sostenible al que los forjados de los edificios modernos deberían aspirar reduciendo al máximo su Carbono Incorporado (CI). Esta tesis sostiene que existe una alternativa para diseñar forjados funcionales manteniendo su naturaleza ligera y esbelta. Esta solución consiste en aumentar su amortiguamiento desde la etapa de diseño. Esto implica, diseñarlos considerando la integración de dispositivos de amortiguación que mejoren su respuesta dinámico, mientras que la masa y la rigidez son empleadas para cumplir con estados límite estáticos.

La investigación realizada en esta tesis ha estudiado una tecnología de amortiguación específica aplicable a pisos compuestos ligeros llamada Amortiguamiento de Capa Constreñida o ACC. Más específicamente, el tratamiento estudiado fue desarrollado por ARUP a principios de la década de 2000 bajo el nombre "resotec" y ha demostrado ser lo suficientemente eficiente como para reducir la respuesta resonante de forjados de gran luz en edificios de oficinas. Este tratamiento consiste en una fina capa viscoelástica (VE) ubicada entre la losa de hormigón y el perfil de acero de una viga mixta. Cuando esta viga vibra a flexión, la capa VE se deforma cíclicamente a rasante disipando energía adicional y aumentando la razón de amortiguamiento del forjado. Este tratamiento suele aplicarse únicamente a lo largo de una proporción de la longitud total de la viga cercana de los apoyos, dejando la región central de ésta conectada a rasante mediante pernos, con el fin de no perder tanta rigidez a la flexión. En consecuencia, esta solución implica diseñar un forjado en el que la acción mixta no se aproveche completamente, pero en el que a cambio se mejora el comportamiento dinámico. A pesar de que esta tecnología ya se ha aplicado en proyectos de construcción reales, su potencial no se ha investigado por completo. Además, pese a su gran utilidad, la aceptación y difusión de esta tecnología no

ha sido la inicialmente esperada. Esto es comprensible, ya que la mayoría de ingenieros actuales no tienen la perspectiva, el conocimiento o las herramientas precisas para saber cuándo esta solución puede ser eficaz y, de ser así, cómo diseñarla o incluso predecir el incremento de amortiguamiento al que se puede aspirar. Esta tesis tiene como objetivo llenar este vacío mediante la realización de un estudio integral de tratamientos ACC aplicados a forjados mixtos. Para ello, se ha estudiado en detalle la modelización de estos tratamientos. Se ha realizado una profunda comparación entre diferentes enfoques de modelización, desde ecuaciones diferenciales en derivada parciales (EDPs) resueltas por tramos (para reproducir vigas tratadas parcialmente) hasta modelos detallados de elementos finitos (EF). El análisis modal ha sido la principal herramienta utilizada para ello. Además, se han realizado ensayos experimentales en tres modelos a escala reducida de vigas mixtas tratadas con ACC. Esto ha permitido comprobar su eficacia y evaluar la precisión de los modelos usados de cara a predecir el incremento de amortiguamiento. Finalmente, se ha realizado un ejemplo de aplicación en el que se han diseñado diferentes forjados mixtos teniendo y sin tener en cuenta el uso de esta tecnología. Para ello, se ha planteado un problema de optimización estructural multiobjetivo. Esta investigación no solo proporciona a los ingenieros herramientas para diseñar estos tratamientos y predecir su comportamiento, sino que también demuestra que en ciertos casos, su uso conduce a estructuras de forjado más ligeras, útiles y sostenibles.

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Abbreviations

ADC	Analog to Digital Converter
AISC	American Institute of Steel Construction
ATMD	Active Tuned Mass Damper
AVMSE	Absolute Value Modal Strain Energy
AVS	Active variable Stiffness
BC	Boundary Conditions
CC	Concrete Centre
CLD	Constrained layer damping
CLT	Cross Laminated Timber
CMA	Complex Modal Analysis
DAC	Digital to Analog Converter
DSLS	Deflection serviceability limit state
DL	Dead load
DLF	Dynamic Loading Factor
EC	Embodied carbon
EMA	Experimental Modal Analysis
EWP	Engineering Wooden Product
FE	Finite Element
FEA	Finite Element Analysis
FRF	Frequency Response Function
FSS	Full Scale Specimen
HFF	High-frequency floors
HIP	Heathcote Industrial Polymers
HIV	Human-Induced Vibrations
HSI	Human-Structure Interaction
IEPE	Integrated Electronic Piezoelectric
HW	Heavy Weight
LL	Live load
LFF	Low-frequency floors
LS	Limit state
LW	Lightweight
LWC	Lightweight Concrete
M1	Model 1 - Detailed FE model
M2	Model 2 - Simplified FE model

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M3	Model 3 - Analytical model
MSE	Modal Strain Energy
MTVV	Maximum Transient Vibration Value
NI	National Instruments
NA	National Annex
OF	Objective Function
OMA	Operational Modal Analysis
PDE	Partial Differential Equation
PT	Pres-stressed
RC	Reinforced Concrete
RKU	Ross Kerwin and Ungar
RMSE	Rongong's Modal Strain Energy
RSS	Reduced Scale Specimen
RW	Regular Weight
SCI	Steel Construction Institute
SDOF	Single Degree Of Freedom
SF	Safety Factor
STMD	Semi-active Tuned Mass Damper
TMD	Tuned Mass Damper
UB	Universal Beams
ULD	Unconstrained Layer Damping
ULS	Ultimate Limit State
ULW	Ultra-Lightweight
VE	Viscoelastic
VC	Vibration Criteria
VDV	Vibration Dose Value
VSL	Vibration Serviceability Limit State

Nomenclature

$A_{st,i}$	Area of the steel profile of beam ' i '
$A_{slab,i}$	Area of the slab section belonging to the section of the composite beam ' i '
a_g	Acceleration of gravity
$a_{p,imp}$	Impulsive peak acceleration
$a_{rms,res}$	Root mean square resonant acceleration
$a_{rms,imp}$	Root mean square impulsive acceleration
B	Total width of the floor parallel to the primary beams
$B_{eff,i}$	Effective width of the area involved in the fundamental mode of vibration of beams ' i '
$b_{eff,i}$	Effective breadth of the slab of the composite beam ' i '
$b_{f,i}$	Width of the top steel flange of the beam ' i '
$b_{v,i}$	Width of the VE core of the CLD treatment applied to the beam ' i '
C_1	Calibration coefficient
C_2	Calibration coefficient
$DSLS$	Functions to compute the DSLS of primary and secondary floor beams
d_2	Separation between secondary beams
E_c	Young modulus of concrete
E	Law of Young modulus along the beam ' i '
E_{st}	Young modulus of steel
EC	Embodied carbon per unit of area of the floor
F_h	Human-induced dynamic punctual force
f_n	Fundamental natural frequency of the floor
f_{step}	Pacing frequency of the human walking load
f_u	Ultimate tensile strength
f_y	Steel yielding stress
f_1, f_2	Objective function
G'_v	Storage modulus of the VE material
g	Shear parameter
g_{opt}	Optimum shear parameter
$g_1 - g_6$	Design constraint functions
H	Modified Heaviside function
h_c	Height of the concrete slab over the ribs in cm
$h_{v,i}$	Height of the VE core of the CLD treatment applied to the beam ' i '
$I_{c,i}$	Concrete-Homogenized Moment of the composite section of beam ' i '
I_{eff}	Effective impulse due to human footfall

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$I_{eff,i}$	Effective concrete-homogenized moment of inertia of beam ‘ i ’
I_{slab}	Concrete-homogenized moment of inertia of 1 m of slab
$I_{st,i}$	Moment of inertia of the steel profile of beam ‘ i ’
i	Subscript to indicate the type of beam: 1 for primary and 2 for secondary
K_c	Calibration factor
K_{cre}	Resonant build-up factor
K_2	Calibration factor
L	Total length of the floor parallel to the secondary beams
L_i	Length of the beam ‘ i ’
$L_{v,i}$	Half of the CLD-treated length of a floor beam ‘ i ’
LS_{slab}	Functions to compute the LSs of concrete slab
M	Mass per unit of area of the floor
M_{Ed}	Design sagging bending moment
$M_{Ed,s}$	Service-life design sagging bending moment of a beam
$M_{p,st}$	Plastic bending moment of a steel profile
M_{Rd}	Resisting sagging bending moment
$M_{Rd,comp}$	Resisting sagging bending moment of a composite steel–concrete section
$M_{Rd,st}$	Resisting sagging bending moment of a steel section
$M_{VSLS,i}$	Bending moments law under the loads considered for the VSLS in beam ‘ i ’
N_c	Axial force in the concrete when a partial degree of shear connection is considered.
N_{cf}	Axial force in the concrete when the full degree of shear connection is considered.
N_i	Number of beams of type ‘ i ’ involved in the fundamental vibration mode of the floor
n	Total number of shear studs
P_i	Profile number associated with the beam ‘ i ’
P_{rd}	Shear resisting force of a shear stud
p_c	Crossover probability
p_m	Probability of mutation
Q	Average human weight
$q_{DL,i}$	Load per unit of length in beam ‘ i ’ due to deal load
$q_{LL,i}$	Load per unit of length in beam ‘ i ’ due to live load
$q_{SW,i}$	Load per unit of length in beam ‘ i ’ due to self-Weight
$q_{VSLS,i}$	Load per unit of length in beam ‘ i ’ considered for the VSLS
R_d	Number to designate the rib-deck
R_{imp}	Impulsive response factor
R_{res}	Resonant response factor
$SF_{M+,c}$	Safety factor for sagging bending moment of the slab in construction
$SF_{M-,c}$	Safety factor for hogging bending moment of the slab in construction
$SF_{M+,s}$	Safety factor for sagging bending moment of the slab in service life
$SF_{M-,s}$	Safety factor for hogging bending moment of the slab in service life
$SF_{M+,c,i}$	Safety factor for sagging bending moment in the construction of beam ‘ i ’
$SF_{M+,s,i}$	Safety factor for sagging bending moment at mid-span in the service life of beam ‘ i ’

$SF_{M+B,s,i}$	Safety factor for sagging bending moment at section B in the service life of beam ‘ i ’
$SF_{Rmax,s}$	Safety factor of VSLs considering impulsive and resonant floor response.
$SF_{Rres,s}$	Safety factor of VSLs considering only resonant floor response.
$SF_{V,c}$	Safety factor for shear of the slab in construction
$SF_{V,s}$	Safety factor for shear of the slab in service life
$SF_{V,c,i}$	Safety factor for shear in construction of beam ‘ i ’
$SF_{\delta LL,s}$	Safety factor for deflection of the slab in service life
$SF_{\delta LL,s,i}$	Safety factor for deflection at mid-span of beam ‘ i ’
$SF_{\delta SW,c}$	Safety factor for deflection of the slab in construction
s	Longitudinal spatial coordinate along a beam element
t	Time variable
$U_{bend,i}$	Modal strain energy of bending of a beam ‘ i ’
ULS	Functions to compute the ULS of primary and secondary floor beams
$V_{p,st}$	Plastic shear force of the web of a steel profile
$VSLs$	Function to compute the VSLs of the floor
W_{eff}	Effective weight associated with the fundamental mode of vibration of a floor bay
$W_{eff,i}$	Effective weight associated with the fundamental mode of vibration of beam ‘ i ’
\underline{x}	Vector of design variables
Y	Geometric parameter of a CLD-treated beam
α_h	Dynamic loading factor
$\Delta\xi$	Additional damping ratio
δ_i	Maximum static deflection of beam ‘ i ’ under the loads considered for the VSLs
η_c	Parameter of crossover
η_m	Parameter of mutation
η_v	Loss factor of the VE material
λ_i	Parts per unit of CLD-treated length in a beam with subscript ‘ i ’
ξ_n	Modal damping ratio of the fundamental mode of vibration
ξ_{int}	Damping ratio of the floor due to its intrinsic energy dissipation capacity
ξ_{CLD}	Additional modal damping ratio of the floor provided by the CLD treatment
$\xi_{CLD,i}$	Additional modal damping ratio provided by the CLD treatment to a beam ‘ i ’
ρ	Density

1

Introduction

1.1 Motivation

The construction industry currently consumes approximately 30% of global material production [1] and is accountable for about 11% of global CO₂ emissions [2]. Moreover, between 2015 and 2021, the global built floor area increased by 11%, and it is expected to grow more than a 40% by 2050 [2]. In this context, the development of more lightweight structural designs emerges as a key point to promote new buildings with minimal environmental impact and reduced Embodied Carbon (EC).

In fact, over recent decades, advances in construction materials and the evolution of sophisticated structural analysis tools have paved the way for addressing more lightweight structural designs. These advancements have enabled engineers not only to design structures that are lighter but also more slender, with longer spans, fewer non-structural components, and higher aesthetic quality (thus, also responding to current architectural trends). However, in this quest to minimize material usage, resulting structures exhibit reduced mass, lower inherent damping, and usually, lower natural frequencies, hence, becoming more susceptible to dynamic loads, such as those induced by humans, machinery, wind, or vehicles. For these reasons, the sizing of some of these structures, especially the lighter ones, is becoming frequently governed by their dynamic response within the framework of the Vibration Serviceability Limit State (VSLS). In this sense, the development of serviceable structures in terms of vibration that meet current green credentials is a current challenge in the structural engineering field.

Human-induced vibrations (HIVs) represent an increasingly concerning serviceability issue for the design of modern pedestrian structures, such as footbridges and building floors. The dynamic loads produced by humans when walking, running or performing rhythmic activities have a periodic nature composed of several harmonics of decreasing energy, all integer multiples of a fundamental one (the most energetic) around 2 Hz, which is regular walking frequency. These harmonics may induce resonant dynamic responses in these structures that if excessive may affect the comfort of their users. Structural designers are aware of this problem and indeed, it is widely known that footbridges' vertical natural frequencies should be above 2.1 Hz (to avoid resonances with walking humans), or that lightweight floors' natural frequencies should exceed the value of 8 Hz (to avoid annoying resonances caused by up to the 4th harmonic of the walking load).

On the one hand, vibrations in footbridges, although deeply researched after the lateral synchronization event of the Millennium Bridge in the year 2000, have a limited impact on people's life, as not everybody daily crosses a footbridge and because walking humans are much more tolerant to vibrations (accepting acceleration levels from 0.5 to 2.5 m/s²) compared to still ones. On the other hand, the impact of building floor vibrations is much higher as many people live or work in multi-story buildings, most of the time sitting or standing still and thus, being more sensitive to annoying vibrations (in fact, the typical acceleration limit for building floors is around 0.05 m/s²). This thesis is devoted to HIVs on building floors.

Focusing on floors, VSLs has become an important issue for building structural designers. Proof of this is the survey conducted by Pavic [3] to 100 structural practitioners from around the world, in which it is stated that: *'Almost a quarter of respondents had experienced problems with human comfort in designs which were code compliant and over 40% stated that they had experienced limitations in design code, guidance/requirements.'* During recent decades, building floors and especially office ones have experienced a reduction of: i) live loads, of around 1 kN/m² (due to the rise of the electronic-paperless offices), ii) damping ratios, around 1-2% due to the progressive removal of heavy furniture and full-height partitions, iii) structural mass because of the rise of lightweight construction materials as Engineered wooden products (EWPs) or cold-form steel, and iv) natural frequencies, mainly caused by longer spans between columns currently used in modern buildings. Nowadays, when engineers face with the design of a floor prone to vibrate, whether they stiffen the structure to raise its natural frequencies as seen in Figure 1.1.(b) (therefore avoiding resonances with human walking), or they make use of heavy concrete slabs to increase the structural mass and ensure a limited dynamic response of the floor

as depicted in Figure 1.1.(c). These practices lead to significant rises in the material used for the floor (especially relevant when the vibration limits to comply are very restrictive) affecting its lightness and increasing drastically its EC. This has been recently proved by Gonçalves *et al.* [4]. Alternatively, the integration of damping strategies into floors' designs (Figure 1.1.(d)) (this means, including devices or technologies that enable increasing the damping of the floor since the design stage) may be an effective way of improving their dynamic performance without increasing their structural mass, thus, minimizing their EC. This is a key improvement considering that floors are already responsible for around 40% of the total EC of a building structure, as can be seen in Figure 1.2.

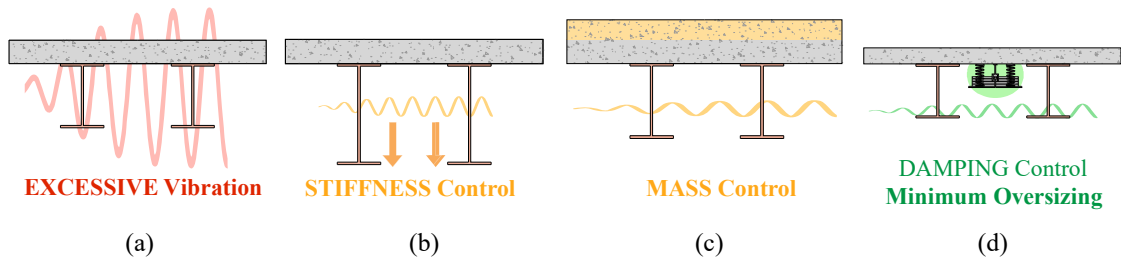


Figure 1.1: Design approaches adopted to develop floors with an adequate dynamic performance in terms of HIVs.

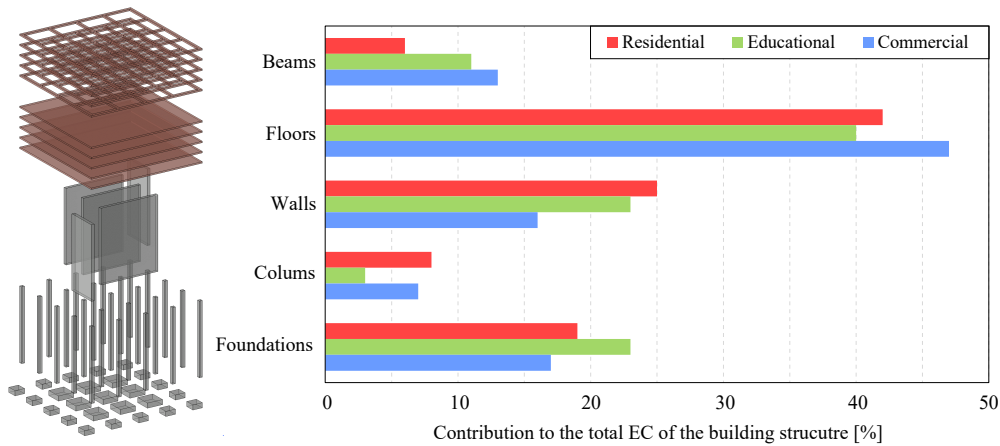


Figure 1.2: Contribution of each structural element to the total EC of the structure of a building. [5]

This design philosophy is based on increasing the structure's damping (instead of the mass or the stiffness) and it is applicable to those structures with a sizing determined by its resonant dynamic response. This damping-integrated design approach could be seen as a particular category within the broader field of 'Motion-based design' developed by Connor [6] in the late '90s.

1.1.1 Introductory case of study

To better illustrate this damping-integrated design approach, an introductory case of study in which it is applied to the sizing of a lively footbridge, is here described. Although this thesis is related to building floors, this example serves as a simplified and knowledgeable version of the concepts later discussed in the document.

A simply-supported box-girder composite footbridge with a main span L of 30 m and a width of 1.5 m has been designed according to Eurocode 3. *Hivoss Design of footbridges guideline* has been used to assess the VSLs. Three main requirements have been established for a code-compliant design: i) the Ultimate limit state (ULS) of bending and shear just considering the steel section, ii) a Deflection SLS under an active live load of 5 kN/m² equal to $L/350$ and iii) a VSLs defined by a maximum peak acceleration of 0.5 m/s² when a stream of pedestrians with a density of 0.5 people/m² is crossing the bridge. Furthermore, the concrete slab has been maintained at a minimum height of 10 cm to minimize the weight of the footbridge (hence, easing its positioning in the construction process using a crane). The thicknesses of the flange and web plates have been kept constant (their dimensions can be found in Figure 1.4) and the height of the web, h_w , has been optimized to optimally size the footbridge.

Two design approaches conceptually depicted in Figure 1.3 have been adopted. First, a traditional one, which is based on stiffening the structure (increasing the web height) so its vertical natural frequency does not lay in the critical range around 2 Hz, and thus the vibration is minimized. Second, a damping-integrated design approach is applied by including a tuned vibration absorber (most commonly known as a tuned mass damper or TMD) at the mid-span section inside the box girder. Thus the box-girder section is designed to comply with static limit states whereas the VSLs is addressed with the TMD (which has been optimally designed according to the *SAMCO-2006: Guidelines for structural control*).

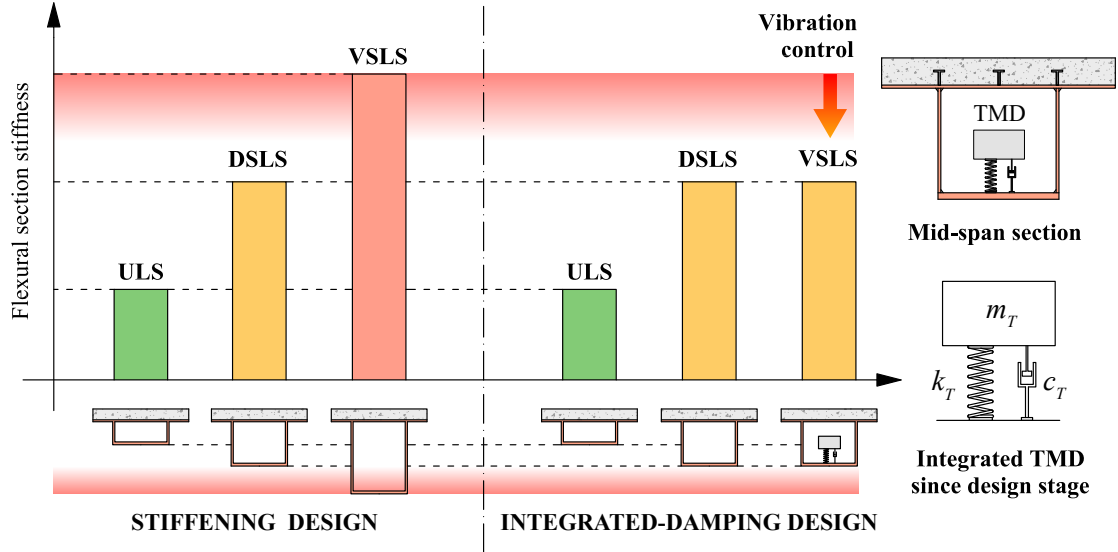


Figure 1.3: Comparison of a traditional design approach with an integrated-damping design applied to a footbridge.

The results of this comparison are summarized in a table attached within Figure 1.4. There, the different design ratios of each limit state are included, additionally, the slenderness L/h_t (where h_t is the total height) and the total weight W of the final designs have been included. Figure 1.4 itself depicts the steady-state acceleration response of each footbridge design depending on the pacing frequency, f_p , adopted by the stream of pedestrians considered (which is modeled as a uniformly distributed harmonic load). Moreover, the figure includes a shaded area representing the amplitude of the dynamic walking force that would induce a vertical resonant response on the footbridge, revealing that designs with vertical natural frequencies within 1.5 to 2.3 Hz are much more prone to vibrate.

In this case, the regular design approach has provided a design with a vertical natural frequency of 2.8 Hz, a web height of 740 mm and 35% of sectional utilization in ULS. The integrated-damping design leads to a much more flexible and slender structure, with a natural frequency of 1.7 Hz, a web height of 415 mm and a higher sectional utilization of 62%. This sectional reduction is achieved on account of the integration of a TMD with a moving mass of 146 kg. In the end, the amount of material saved totals 2200 kg (which represents a 10% decrease in the structural steel used).

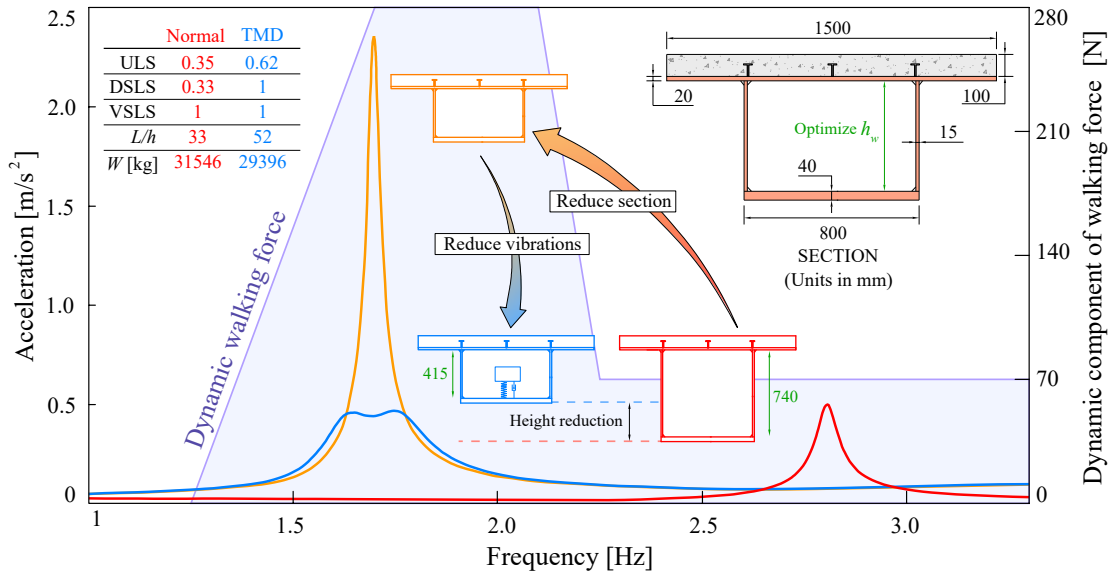


Figure 1.4: Dynamic performance as a function of the pedestrian pacing frequency of the footbridge designs here compared.

This basic example, developed in the early stages of this doctoral thesis, encouraged the author to explore the benefits of integrating damping technologies in the design of structures subjected to dynamic loads for further optimizing their layout while enhancing their vibration performance.

1.1.2 Damping technology studied: Constrained Layer Damping

This doctoral thesis is devoted to investigating the potential of applying the above-explained integrated-damping approach to the design of lightweight composite floors. However, a different damping technology from TMDs has been investigated (as inertial control techniques have already been deeply researched in the past, so their working and design principles are nowadays consolidated). Instead, a dissipative damping technique has been studied, and more specifically one based on 'constrained layer damping' (CLD) principles.

CLD treatments consist of a thin layer of a high-damping material, usually viscoelastic (VE), embedded between two elastic bending members. This creates a sort of 'sandwich beam' with a VE core that deforms greatly to shear when the beam vibrates in bending modes, hence dissipating additional energy through a VE stress-strain shear hysteresis. CLD treatments were developed and have been extensively applied in aerospace and

mechanical engineering applications to mitigate broad-band frequency vibrations [7]. Nevertheless, their use in civil engineering is still limited.

In 2006, the engineering consultancy company ARUP, in collaboration with Richard Lee Steel Decking proposed the first commercial CLD solution to be integrated into lightweight composite floors since the construction stage. This was called 'Resotec' and its implementation was described by Willford *et al.* [8] (Figure 1.5). 'Resotec' emerged as a project remedy when the developer of the More London Plot 1A building (depicted in Figure 1.6) decided to double the span of the column grid passing from 11.25 m to 22.5 m. Although the new design met the VSLs limitations usually applicable to office floors at that time, the client was advised that there was evidence of increased office worker sensitivity to floor vibrations. At that point, it was decided to investigate the possibility of a cost-effective method of significantly reducing floor vibrations by providing additional damping to the floor without incurring in significant floor oversizing.

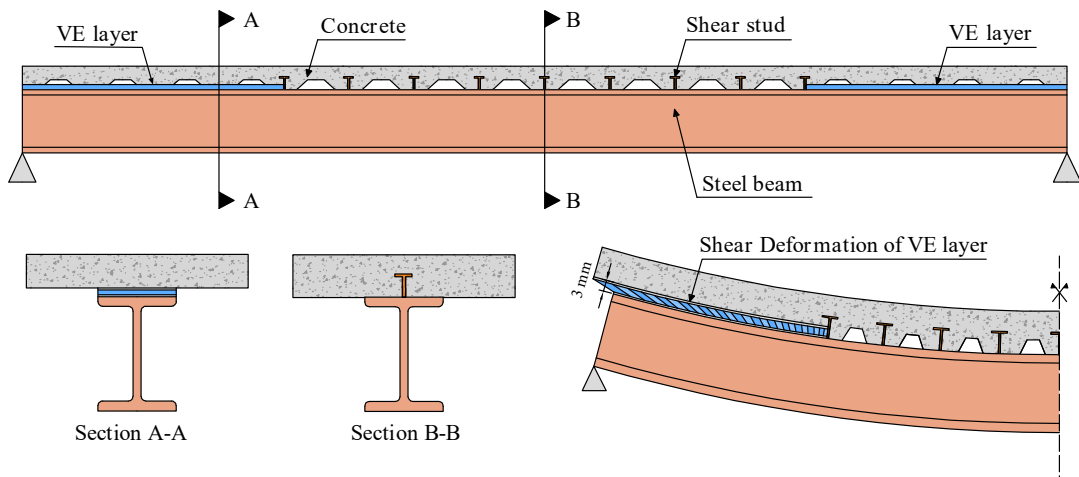


Figure 1.5: 'Resotec' CLD treatment installed along 50% of the beam's length (50% CLD) Willford et al. [8].

In ARUP's proposal (depicted in Figure 1.5 and in Figure 1.7), a thin VE layer was located between the rib-deck concrete slab and the upper flange of the steel profile of the composite beams (where the shear strain to be achieved is maximum). Moreover, they included the VE layer only for a percentage of the beam's length near the supports, where the longitudinal shear strain is higher, whereas the central part of the composite beam remained connected to the longitudinal shear using studs. Therefore, their proposal emerged as a trade-off solution in which a loss in bending stiffness and capacity is granted in return for a higher damping ratio of the floor. In fact, in Figure 1.8, it can be clearly

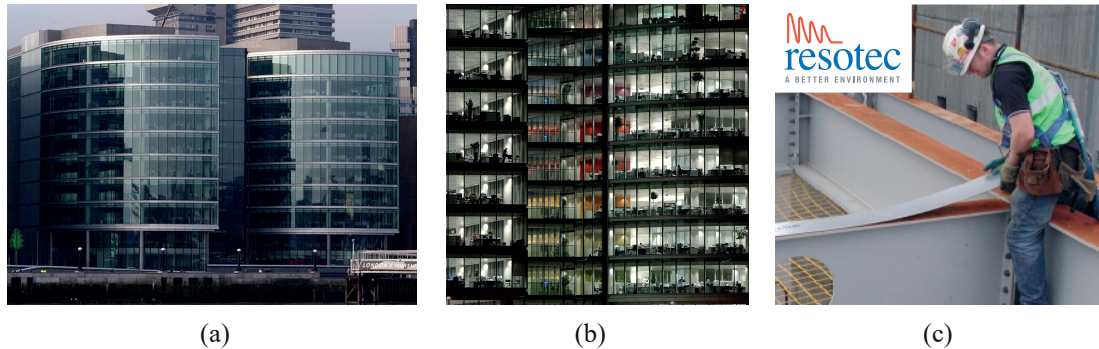


Figure 1.6: (a) First project in which 'Resotec' CLD treatment was integrated into the floor design, the More London Plot 1A building. (b) night view of the office floors of the More London Plot 1A. (c) Application of 'Resotec' during the construction process.

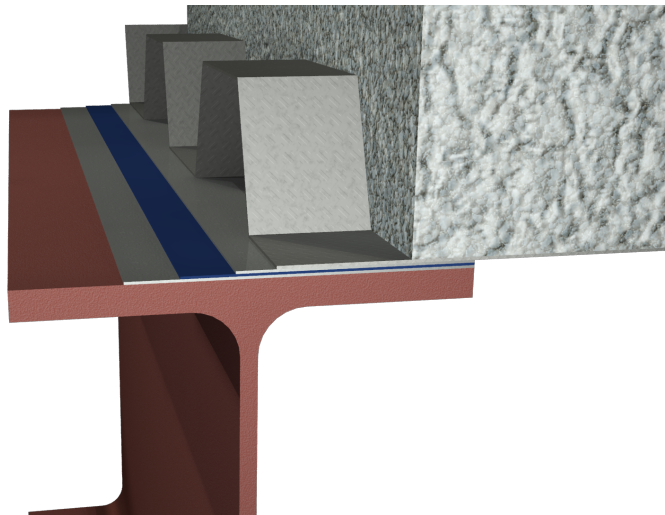


Figure 1.7: Perspective of the VE CLD treatment studied

appreciated that the CLD-treated beam tested by Willford *et al.* [8] has a lower natural frequency but a higher damping ratio than the fully composite untreated one. The CLD treatment itself proposed by ARUP (the one depicted while being installed in Figure 1.6.(c)) has an overall thickness of 3 mm and it is composed of two thin steel sheets of 1 mm that constrain a slim VE layer of 1 mm. According to Willford *et al.* [8], no adhesive is needed to fix the treatment to the steel profile or the slab «[...] as frictional resistance is sufficient to provide the necessary load transfer for the very small dynamic strains resulting from footfall excitation».

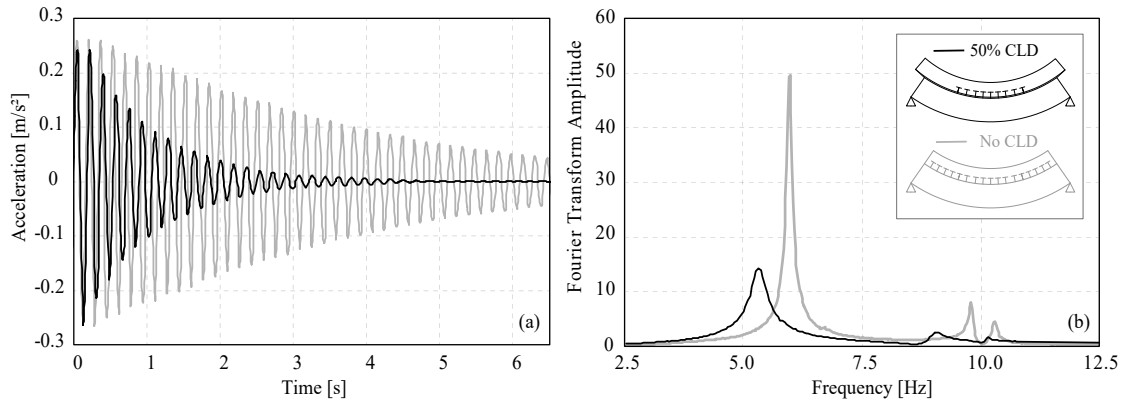


Figure 1.8: Free decaying response of two composite beams tested by Willford *et al.*[8] with and without a 50% CLD treatment. (a) Time domain. (b) Frequency Domain [8].

Hence, CLD treatments have been satisfactorily applied by ARUP in some building projects to avoid excessive structural oversizing just to control HIVs. Nevertheless, their optimal arrangement and integration into the design workflow of composite floors and their potential to provide more lightweight, sustainable and comfortable structures are aspects not yet deeply researched. This doctoral thesis aims to fill this gap meeting the objectives described in the following section.

1.2 Objectives

The design of lightweight composite floors is increasingly conditioned by their dynamic response when subjected to human footfall loading. In these structures, the amount of additional stiffness or mass needed to meet this VSLs requirement results in being unreasonably high. This has prompted engineers to explore other design approaches focused on enhancing floors' damping capacity. In this respect, the integration of VE CLD treatments into floor designs has been satisfactorily applied in certain building projects obtaining an improvement in their dynamic performance without incurring severe structural oversizing. Despite this fact, the use of CLD treatments did not spread as expected. Indeed, to date, there is still a lack of applied knowledge for practitioners to effectively apply and integrate this technology into floor designs.

In this context, the main aim of this doctoral thesis is to develop a deep study of the design of lightweight composite floors with integrated CLD treatments. In doing so,

the author aspires to provide the necessary perspective, knowledge and tools for building engineers to study the viability of this solution and if deemed necessary, to accurately and efficiently integrate it into future lightweight floor designs.

With this in mind, the specific objectives of this thesis are the following:

- The development of an updated and historically contrasted review of the current state-of-the-art of the VSLS on building floors under Human-induced excitation. This includes providing a classification of vibration control techniques currently available to enhance the dynamic performance of floors.
- To extend the analytical solutions currently available for the modal properties of the so-called 'sandwich beam' with VE core. An analytical solution is still needed for cases where the VE core is only present along certain regions of the beam, while in others the outer elastic layers remain connected to longitudinal shear. This will provide an exact and robust solution for beams as the one depicted in Figure 1.5.
- To explore and compare the different alternatives currently available to numerically model a VE CLD treatment with FEA. In doing so, the thesis aspires to provide a simplified and practitioner-focused modelling methodology with results validated and contrasted against more complex modelling approaches.
- The creation of experimental tests on real CLD-treated composite beams to validate the improved dynamic performance achieved by the treatment, and then, compare the experimental findings with the modal parameters previously predicted from analytical and numerical modelling.
- The formulation and solving of a multi-objective structural optimization problem for composite floors with integrated CLD treatments in terms of structural mass and EC. The final objective is to identify those project conditions in which a CLD-treated floor may be an attractive and competitive solution. Additionally, it is pursued to quantify the potential material savings that could be obtained by integrating CLD treatments into floor designs when compared to floor sized without them.

Overall, this research significantly contributes to the field of VSLS of building floors, by addressing a vibration control technique that has not been extensively researched and integrating it into the design of lightweight composite floors. The findings obtained in this work establish a reference for future studies related to the use of CLD treatments for enhancing the dynamic performance of civil engineering structures.

1.3 Outline of the Document

The thesis has been structured with the aim of providing a comprehensive understanding of three elements: i) the issue of HIVs in building floors, ii) the functioning of CLD treatment to mitigate these vibrations, and iii) how to improve lightweight floors performance by integrating this damping technology since their design stage. To accomplish this goal the work has been divided into seven parts:

- Chapter 1: Introduction

This chapter provides an overview of the topic treated in the thesis. First, a motivation for the research is given supported by a brief case study and a short description of the CLD technology studied is performed. Then, the investigation's main objectives are outlined to finish with the document organization.

- Chapter 2: State of the art

The second chapter of the thesis provides a thorough review of the current state of knowledge in the VSLs of building floors. First, the assessment of VSLs due to HIVs is deeply discussed from the first methodologies initially developed till the latest research on the field. Then, a detailed classification of the various vibration control techniques currently applicable to floors is presented, to finally focus on the VE CLD treatments covered by the thesis.

- Chapter 3: CLD models: Analytical models

This chapter describes the partial differential equation (PDE) of the 'sandwich beam' with a VE core. A numerical methodology for obtaining exact complex modal properties of three-section beams (with a central region rigidly connected to longitudinal shear and two lateral regions VE-treated), is here provided. The Chapter finishes with the results of a dimensionless parametric study for partially-treated beams.

- Chapter 4: CLD models: Numerical models

The fourth chapter of the thesis studies two different approaches to be adopted for FE modelling a CLD treatment. The first is more detailed and uses hexahedral solid elements combined with complex modal analysis (CMA). The second is a practitioner-focused model that reproduces the VE layer with equivalent spring elements and makes use of a modified modal strain energy (MSE) method. Both approaches are used to predict the modal properties of a set of CLD-treated

composite beams, concluding that the simple one is accurate enough to be used at the design stage of the treatment.

- Chapter 5: Experimental tests on reduced-scale models

This chapter describes the design, construction and testing of 3 reduced-scale composite beams of 3.6 m span: i) A fully treated one (100% CLD) ii) one treated along half of its length (50%CLD), and iii) an untreated (0%CLD). The results in terms of natural frequencies and modal damping ratios have been compared with those predicted by the analytical model described in Chapter 3.

- Chapter 6: Application example: Structural optimization

After having discussed the functioning principles of CLD treatments, the different methods available to predict their behaviour and their actual dynamic performance, this chapter addresses their integration into the design process of composite floors. For that, a structural optimization problem is proposed by illustrating all the limits state to be met by the floor and the methodologies to be used for checking them. This problem has been parametrically solved for different floor spans and vibration limits. Finally, a set of useful results has been derived that allows quantifying the improvement in terms of mass and EC provided by the integration of the CLD treatment within the designs. In addition, these results aim to serve as a first guide for engineers keen to assess the applicability of CLD treatments to a particular project.

- Chapter 7: Conclusions

The final chapter of the thesis encompasses the summary of the main findings, conclusions and contributions. In addition, the extent to which objectives have been reached is assessed and future research lines are proposed.

2

State of the art

2.1 Introduction

This chapter covers a literature review of the different topics treated in the thesis. First, the concept of 'lightweight floors' is described, and an accurate definition for it is proposed. Second, a literature review of the research field of floor vibrations is developed, describing the main challenges in this field for the future. Third, a review of all the vibration control technologies available nowadays, and potentially applicable for mitigating floor vibrations, is carried out. Fourth, a description of the mechanical behaviour of VE materials, used in this thesis, is provided. Finally, a brief description of the CLD treatments studied in this thesis is given, as this one is the technology studied in this thesis for its integration into the design workflow of composite floors.

2.2 Lightweight floors

Regarding the structure of a regular multistory building, the floors generally store up to 40% of its total EC. Aware of this, structural engineers are increasingly developing and using lightweight and slender floor systems that allow bridging longer spans with less material. This not only implies a material saving in the horizontal structure, but also, in the vertical one, decreasing the size of columns and foundations.

Lightweight floors display great advantages from a structural efficiency point of view; however, their dynamic performance in serviceability conditions is often unacceptable mainly due to their lower mass. Thus, most of the recent research related to these structures deals with their dynamic behaviour within the VSLs framework, since it has become the critical limit state.

In this subsection, a qualitative definition of the concept 'lightweight floors' will be provided, and the main advances in this engineering field will be commented.

2.2.1 Definition of lightweight floor

The term 'lightweight floor' has been qualitatively used in the literature for referring to those floors with a self-weight per m^2 considerably lower than the one of regular floors usually used in the construction industry. Prior to elaborating further on the definition, it is necessary to introduce the most common flooring systems widely used nowadays, which are depicted in Figure 2.1.

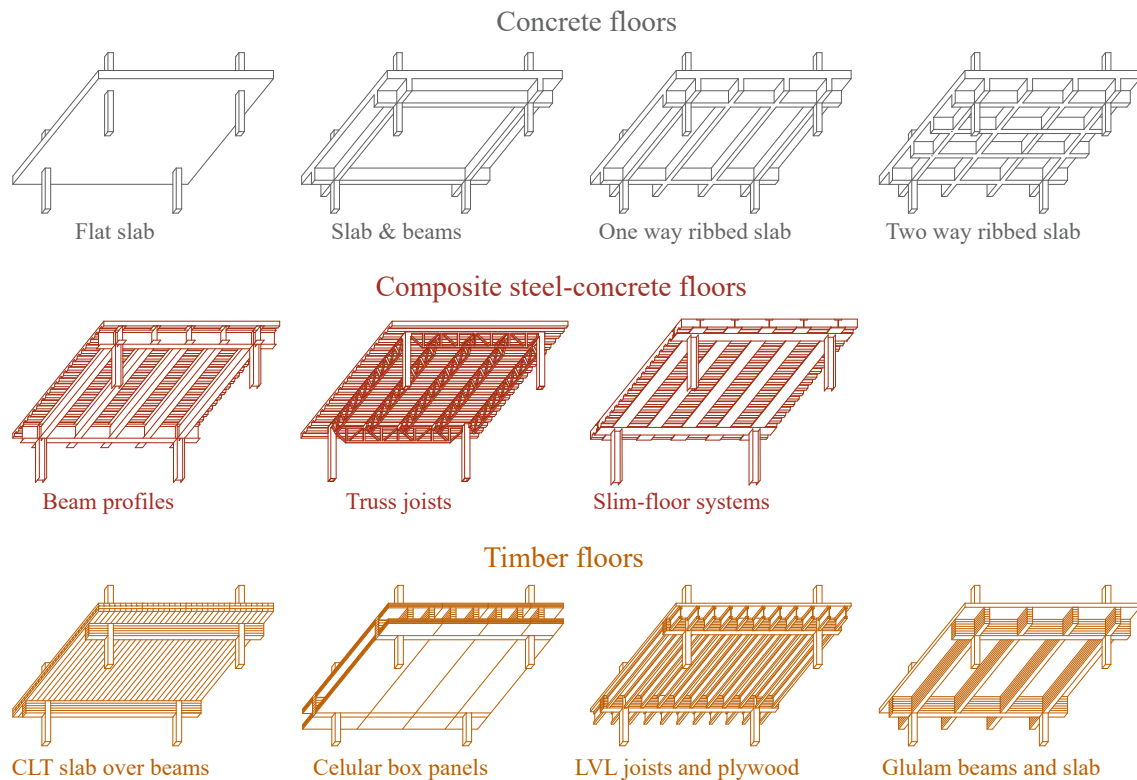


Figure 2.1: Flooring systems most commonly used nowadays in the building construction industry.

Floor systems can be classified depending on their structural material, and in this sense, three main groups arise: i) concrete floors, ii) composite steel-concrete floors, and iii) timber floors. The scheme provided in Figure 2.1 serves as a conceptual classification. Concrete floor systems based on flat slabs or on slabs with beams can be seen as standard floor systems for multistory buildings due to their simple formwork, architectural flexibility and reduced depth. Floors based on timber beams are also quite used for residential two-story buildings in northern countries (as U.S.A., U.K. or Scandinavian countries). Finally, composite steel-concrete floors are mainly used for multistory or high-rise buildings built in cities (especially in those with more than 10 stories) [9].

Concrete floors can be further classified according to two additional criteria: depending on whether they make use of reinforced (RC) or pre-stressed (PT) concrete, or whether they are in-situ cast or precast. It is common to use PT concrete for long-span floors while RC is reserved for low-medium span floors.

Within the literature, the term 'lightweight floor' has been used to refer to composite or timber floors, or to describe an innovative concrete floor system in which the material used has been minimized, regardless of the technique used for that. In order to shed light on this definition, a chart considering the weight of representative floor types has been developed in this thesis based on own developments and the following references: RC Flat slab [10], [11]; PT Flat slab [10], One-way RC [10]. This chart is depicted in Figure 2.2 and includes 6 floor types: 3 concrete floors, 1 composite and 2 timber floors. The floors have been optimally designed considering a dead load of 1 kN/m^2 combined with a live load of 3 kN/m^2 . Besides, deflection limits have been considered but not vibration ones. It is worth noticing that, in Figure 2.2, the continuous lines define the floors' range of use. Hence, the dashed lines reflect the floor sizing for span out of competitive ranges.

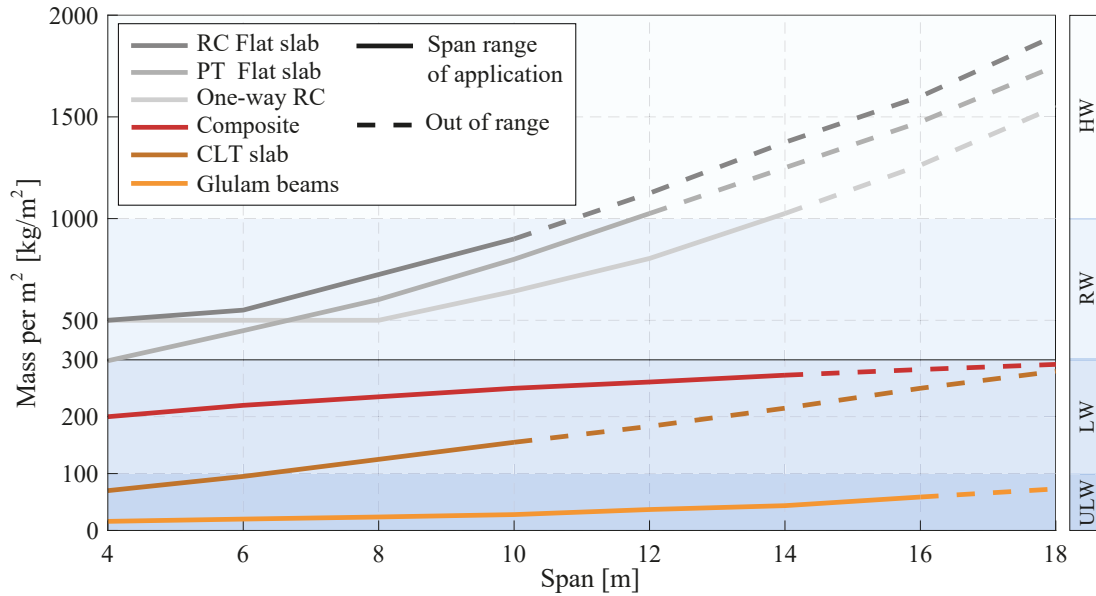


Figure 2.2: Mass per unit of area of representative floor types used in the industry with respect to span value. Blue shadings represent the 4 types of floors depending on the weight. Data Sources: RC Flat slab [10], [11]; PT Flat slab [10], One-way RC [10]; Composite (own development); CLT slab (own development); Glulam beams [12].

From Figure 2.2, it can be extracted that concrete floors are the heaviest followed by composite and timber ones. In addition, the use of beams or ribs is a common strategy to reduce floor weight, something clearly evident when comparing CLT slab floors with timber floors based on Glulam beams. Hence, according to Figure 2.2, 4 classes of floors might be defined in weight terms:

- Ultra-lightweight floors (ULW) with less than 100 kg/m^2
- Lightweight floors (LW) between 100 to $300\text{-}500\text{ kg/m}^2$
- Regular weight floors (RW) between $300\text{-}500$ to 1000 kg/m^2
- Heavy weight (HW) floors with more than 1000 kg/m^2

This analysis confirms that the use of the term 'lightweight floor' to refer to composite or timber floors is adequate. Thus, this definition will be assumed along the whole thesis. In fact, most of the VSLs problems reported in the literature in the last decades are related to this type of floors. Thus, this doctoral thesis is focused on studying a specific damping treatment to be applied to composite floors for solving their VSLs problems. The treatment modelling approach can be straightforward extrapolated to any other

lightweight floor system composed by beams and a relatively rigid slab (i.e. a timber floor composed of Glulam beams and CLT).

2.2.2 Innovative lightweight floor systems

The need to develop effective and feasible damping techniques for application in lightweight floors is driven by the increasing number of innovative floor systems that are emerging today, and that are a response to the need of buildings with less EC. Figure 2.3 provides a summary of some of the innovative lightweight floors recently proposed. These systems are mainly based on three concepts:

- The efficient structural systems as domes [11], vaults [13], trusses [14] or optimized grillages [15].
- The use of sophisticated materials as mortars, expanded foams [14] or mineral wool to lighten the structure while keeping a good sound insulation [16].
- The combination of different structural materials in the same floor [17], [18].

All these floors must have an adequate dynamic behaviour, hence, a human-induced vibration (HIV) assessment is required for them. Indeed, most of the research related to lightweight floors deals with this topic. As a good example of this warning, authors as Hawkins *et al.* and Hossell *et al.* include a granular infill to provide additional mass and damping to the floor so addressing the vibration problem.

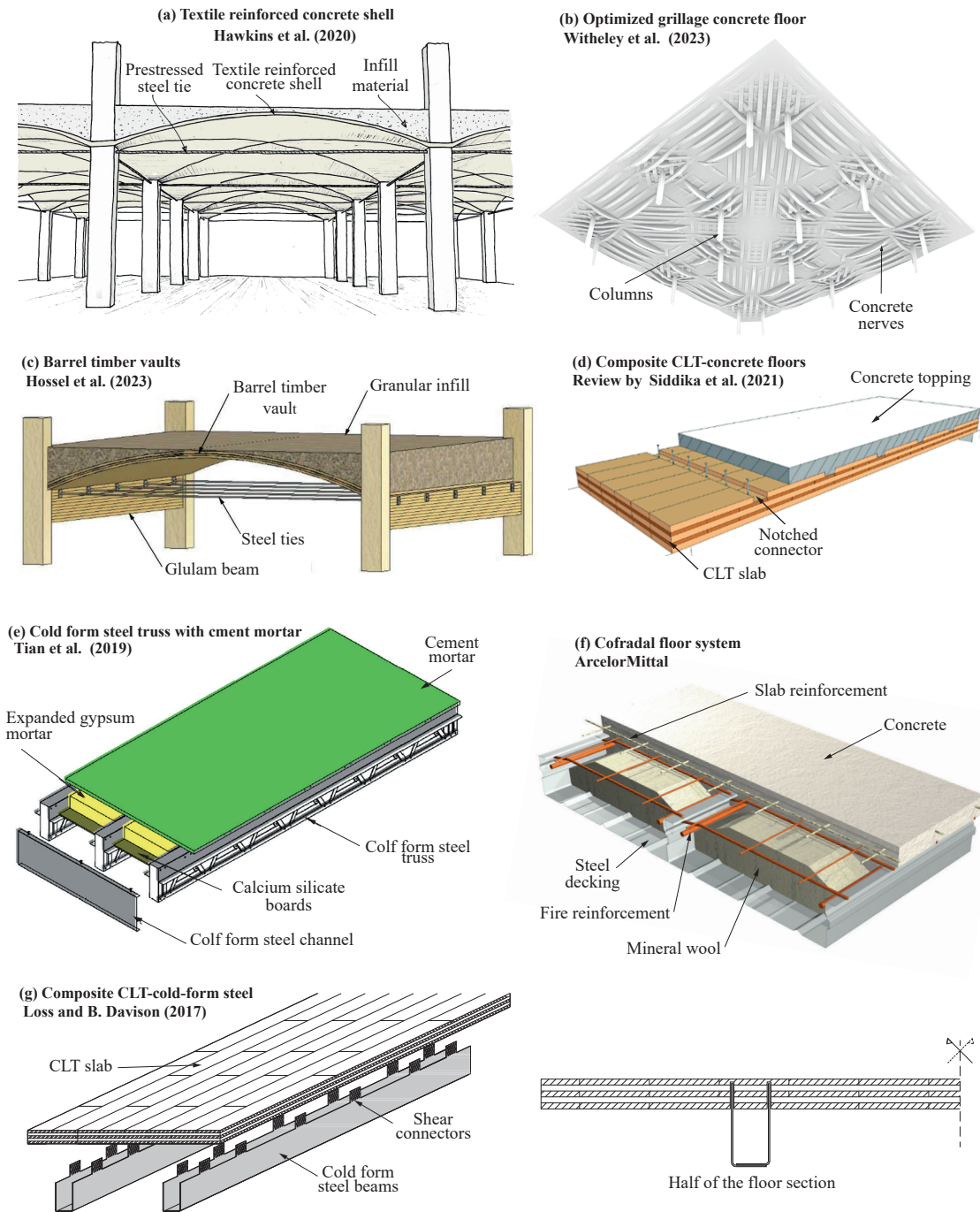


Figure 2.3: Recent innovations in lightweight floor systems. (a) Textile reinforced concrete shells [11]. (b) Optimized grillage concrete floor [15]. (c) Barrel timber vaults [13]. (d) Composite CLT-concrete floors [17]. (e) Cold-form truss floors [14]. (f) Cofradal system [16]. (g) Composite CLT-cold-form steel [18].

2.3 VSLS of floors under human-induced excitation

A VSLS problem is composed by three main constitutive elements: i) a vibration source made of an exciting force at an specific location, ii) a path of vibration, and iii) a vibration receiver.

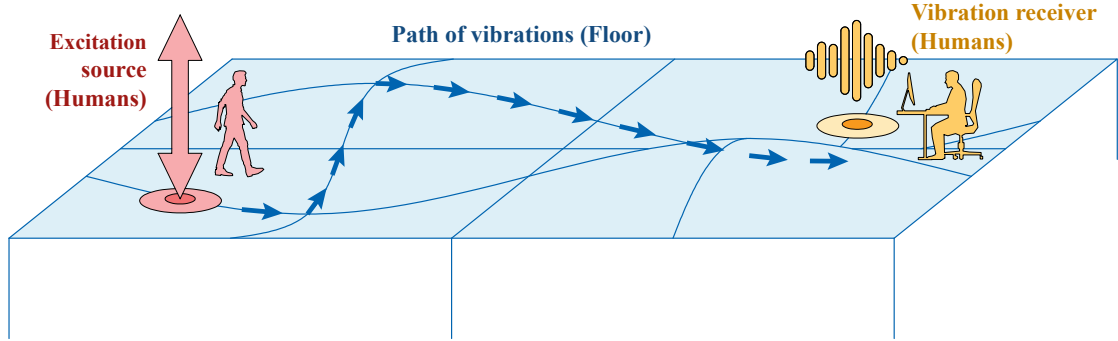


Figure 2.4: Three elements that conform a VSLS problem.

First, the vibration source when dealing with VSLS can come from: wind **aa**, machinery **bb**, vehicles **cc**, and humans **dd**. Focusing on humans, different actions may be the excitation source, for example, walking, running, bouncing, jumping, rhythmic activities, human groups or streams, etc. [19]. This thesis is mainly focused on HIV under walking excitation.

Second, depending on the path of vibrations, the HIV research field can be split into two main groups: i) HIV in footbridges (in lateral and vertical direction) [20], **Setra**, [21], and ii) HIV in floors (mainly focused on vertical direction) [22], [23], [24], [10], [25], [26]. These two VSLS problems mainly differ in their critical frequency band and comfort limits. While critical natural frequencies of footbridges for vertical vibrations are around 2 Hz, in floors they are between 3 to 9 Hz. Additionally, regarding floor structures, it is common to divide them into two groups: i) The so-called 'low-frequency floors' (LFFs) with fundamental natural frequencies, f_1 , lower than 9 Hz (although this threshold could vary depending on the source), and ii) 'high-frequency floors' (HFFs) with f_1 higher than 9 Hz [24]. The dynamic response of these floor groups has a different nature, as will be seen later on.

And third, in a VSLS problem the receiver is a human who is performing a certain activity at a certain location of the structure. The amount of vibration experienced by the human needs to be limited for comfort reasons [27]. These vibration limits, usually established in acceleration terms, are considerably higher in footbridges (from 0.5 to 2.5

m/s^2 [21]) than in building floors (from 0.02 to 0.1 m/s^2 [23]), as humans are more sensitive to vibration in calm indoor ambiances than walking outdoors.

Thus, this thesis deals with the VSLs of floors under human walking excitation in calm ambiances as offices, residences or hospitals. In order to address such a problem, its three defining elements should be accurately characterized: the vibration source, which is the human dynamic excitation; the vibration path, defined by the floor modal parameters, and the vibration receiver defined in terms of comfort limits.

2.3.1 Human excitation source

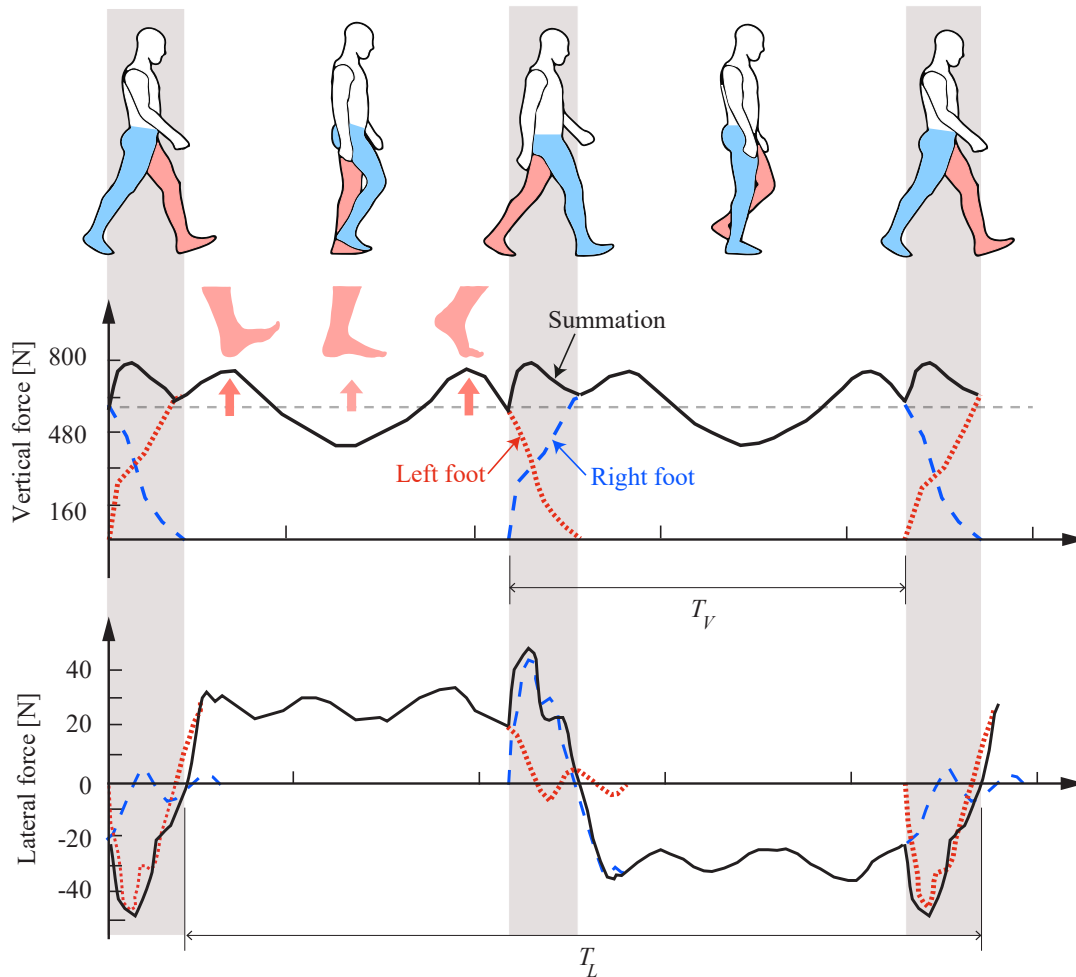


Figure 2.5: Human ground reaction force for walking action in vertical and lateral directions. Figure based on [20] and [28].

Before addressing all the literature related to the VSLs of floors, it is necessary to briefly describe the simplest human-induced excitation: walking.

Figure 2.5 includes a time domain description of this human-induced force in vertical and lateral directions. This force has a periodic nature that can be assumed to be harmonic, with a fundamental vertical period, T_V , around 0.5 s and a lateral one, T_L , around 1 s. The vertical component (the one concerning this thesis) is composed by a constant or static force (equal to the human weight) plus a dynamic component produced by slight impacts of the heel and the toe when making a footfall. When performing the Fourier transform of the vertical component, the amplitude of the frequency spectrum is similar to the one depicted in Figure 2.6.

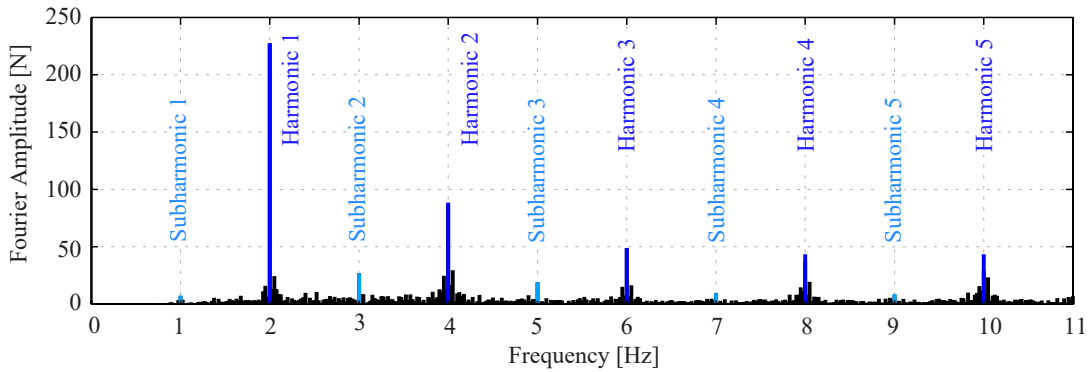


Figure 2.6: Amplitude of the frequency spectrum of the walking ground reaction force in vertical direction. Figure based on [29].

In Figure 2.6, it can be seen that the vertical dynamic walking force is mainly composed by 5 different harmonics (depicted in dark blue). The first one is located at the pacing frequency, f_p , usually around 1.7 to 2.1 Hz. The rest of harmonics are multiples of the first one, being located at $2f_p$, $3f_p$, $4f_p$ and $5f_p$. If the walk is not properly balanced between both feet, so one provides more ground reaction force than the other, some subharmonics between the harmonics may appear (depicted in light blue), however, their influence is negligible.

Once the harmonic nature of the walking load has been introduced, the two types of floors dynamic response to this load can be outlined. LFFs usually exhibit a dynamic behaviour characterized by a steady-state resonant response due to the coupling of one of these walking harmonics (usually the 3rd or the 4th) with a critical vibration mode of the floor (generally the fundamental one). On the contrary, the dynamic response of HFFs does not have a resonant nature, as high-frequency walking harmonics are not energetic

enough to develop it. Instead, a sequence of consecutive floor transient responses to each footfall is obtained. Both dynamic responses are compared in Figure 2.7. For assessing these two types of dynamic response, two models of the walking load have been usually used.

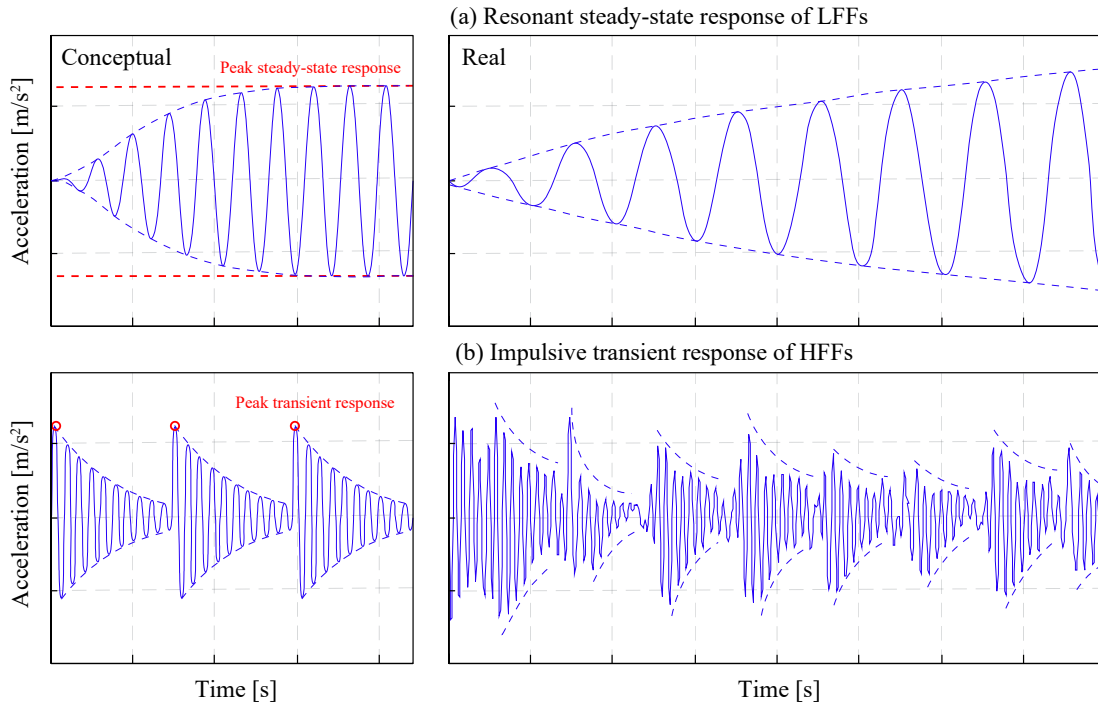


Figure 2.7: Floors dynamic response. (a) LFFs response. (b) HFFs response.

Harmonic model of the vertical walking force for LFFs

Although many strategies have been proposed to model the vertical walking force, F_w , the most widely accepted is the one provided by the following equation:

$$F_w(t) = Q \left(1 + \sum_{n=1}^4 \alpha_n \sin(2 \pi f_p n t + \phi_n) \right) \quad (2.1)$$

where t is the time variable, Q is the human weight (usually around 700 N), n is a natural number representing the harmonic index, α_n is a dynamic loading factor (DLF) that determines the amplitude of each harmonic and ϕ_n is the phase of each harmonic. Harmonics higher than the 4th one are neglected, although recent researches suggest that their influence might be higher than previously thought [30], [31].

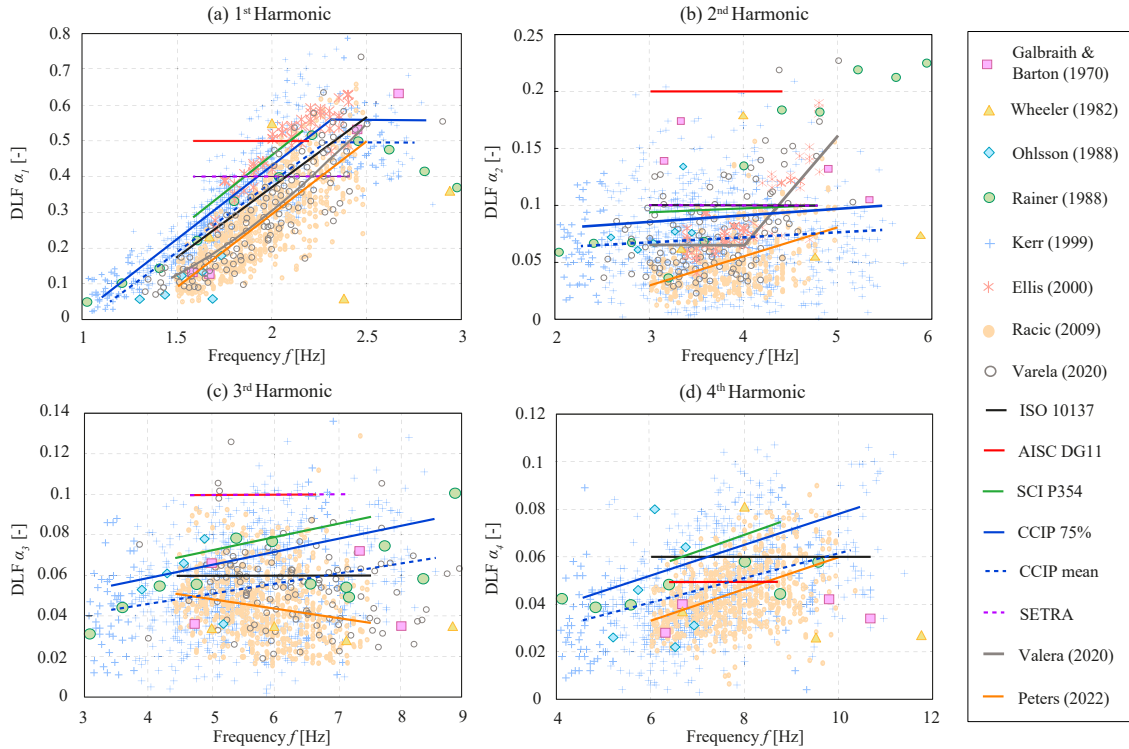


Figure 2.8: DLFs according to different authors and guidelines. (a) DLF of 1st Harmonic. (b) DLF of 2nd Harmonic. (c) DLF of 3rd Harmonic. (d) DLF of 4th Harmonic. Figure modified from [10] with additional information taken from [32] and [33]. References in the legend order: [34], [35], [36], [37], [38], [39], [40], [32], [19], [23], [24], [10], [41], [32], [33].

The amplitude, Q_{α_n} , and the phase, ϕ_n , of each one of these harmonics have been extensively studied in the literature by numerous authors, and so, this is one of the major topics in the HIV research field during the past two decades. Figure 2.9 provides a summary of some of the most relevant works done to characterize DLFs of the walking load. The figure dots represent the different experimental campaigns performed in this respect, and the lines, the models provided by different standards, guidelines and authors. Initially, these models were mere regressions or approximations of the measured reality (as those provided by the ISO10137, SETRA or the AISC DG11), however, more advanced models based on a probabilistic data treatment emerged later (as those proposed by the CCIP or Peters *et al.*).

Some of the design guidelines focused on floor vibrations provide a simplified version to obtain the critical DLF (the one of the harmonic that predominantly causes the resonant floor response) as a function of the fundamental frequency of the floor. Two good examples of this are the exponential functions provided by the AISC Design guideline 11 [23], one

used for within a manual vibration prediction method, and another for Finite Element Models (FEM).

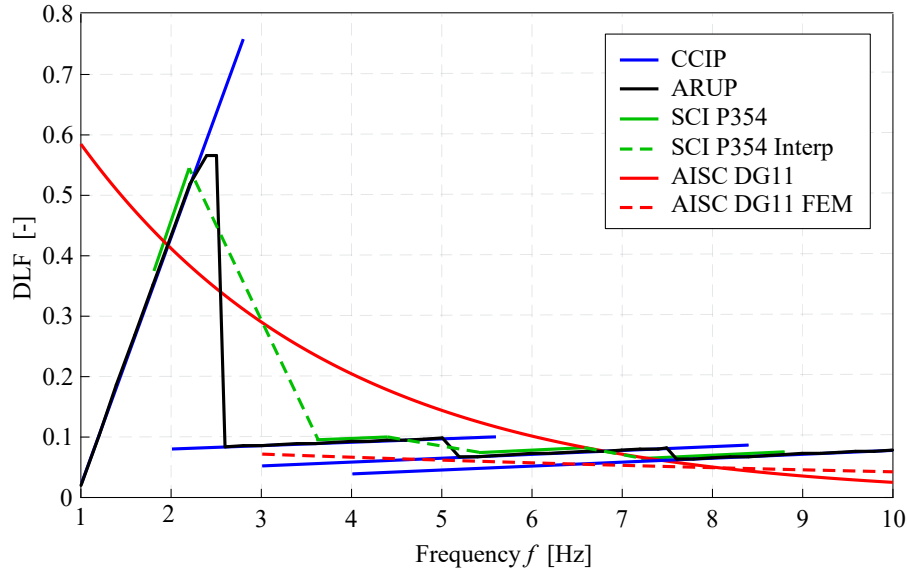


Figure 2.9: DLFs according to different design guidelines for floor vibration. References in the legend order: [10], [42], [24], [23].

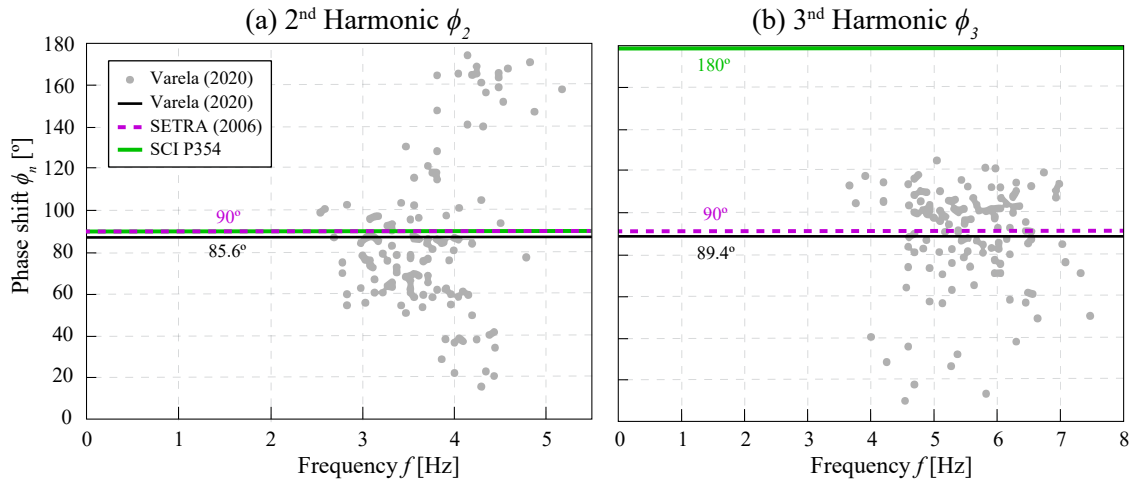


Figure 2.10: Walking harmonics phase shift. (a) 2nd Harmonic. (b) 3rd Harmonic. References in the legend order: [32], [41], [24].

To complete the characterization of the human walking load, the phase shifts ϕ_n need to be defined. These parameters show a much wider dispersion than the DLFs. Figure 2.10 shows a comparison of values provided by different sources, those provided by Boniface, Bui, Bressolette, *et al.* [41] seem to fit better the experimental results. Thus, the following

values could be assumed: $\phi_1 = 0$, $\phi_2 = \pi/2$, and $\phi_3 = \pi/2$.

Effective impulse model of the vertical walking force for HFFs

When assessing the dynamic response of an HFF, the main parameter to be estimated is the peak transient response occurring just after a footfall hits the floor (represented with a red circle in Figure 2.7). This magnitude is inversely proportional to the mass involved in the floor motion and in order to estimate it a model of the footfall called 'Effective impulse', I_{eff} , was developed by Willford *et al.* from ARUP in 2006 [43], [42], [44] [45].

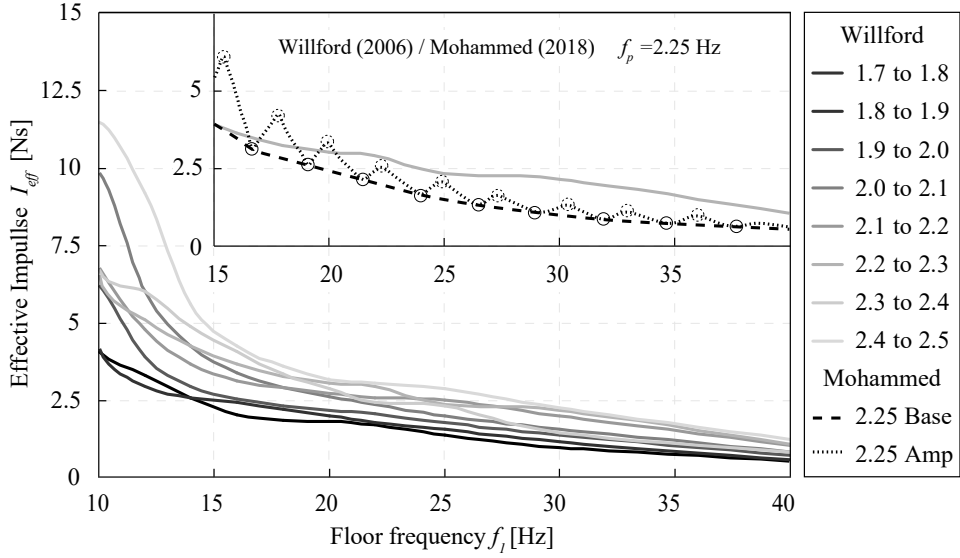


Figure 2.11: Effective Impulse developed by Willford *et al.*

The approach used for deriving I_{eff} was identical to the one used to obtain the 'response spectra' in seismic engineering. Thus, Kerr's footfall forces [38] (with f_p between 1.7 to 2.5 Hz) were fed into a set of SDOF oscillators with unit mass and different f_1 ranging from 10 to 40 Hz. The resulting peak velocity for each combination of $f_1 - f_p$ was taken as the impulse action (see Figure 2.11). Then, two best-fit relationships were extracted for 75 percentile and mean values of I_{eff} :

$$I_{eff} = 54 \frac{f_p^{1.43}}{f_1^{1.3}} \quad (75 \text{ percentile}) \quad (2.2)$$

$$I_{eff} = 42 \frac{f_p^{1.43}}{f_1^{1.3}} \quad (\text{Mean value}) \quad (2.3)$$

The effective impulse proposed by Willford *et al.* is a deterministic model obtained from single footfalls recorded on a force plate. An updated version of this model has been recently proposed by Mohammed *et al.* (a comparison of both models for $f_p = 2.25$ Hz is provided in Figure 2.11). This latter is based on successive footfalls measured on an instrumented treadmill that have been treated with a probabilistic approach. Additionally, it accounts for the higher walking harmonics (the 6th, 7th, 8th... that still can induce resonant effects with higher natural frequencies of the floor) and for the influence of the floor's damping. This is done by building the effective impulse as a product of three probabilistic best-fit components: i) a base function of the impulse (dashed line in Figure 2.11), ii) an amplification factor (represented with a dotted line and applied at multiples of f_p), and iii) a damping correction function.

2.3.2 The floor as vibration path

Within the acceleration ranges typical of vibration serviceability a floor structure mainly behaves linearly, thus, modal analysis can be used to model its dynamic behaviour. Generally, the VSLs of a floor can be assessed in two ways: i) with simplified analytical methods in which it is assumed that the fundamental vibration mode is the only contributor to the floor dynamic response, and ii) with a numeric FE model of the floor.

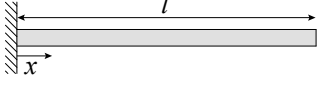

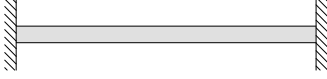
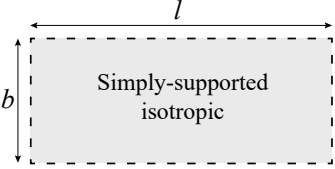
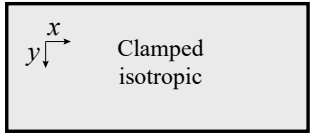

Simplified Analytical models for VSLs

These simplified hand calculation methods are based on estimating the modal parameters of the fundamental mode of vibration of the floor, namely, its natural frequency f_1 , its modal mass M_1 (better named, 'equivalent mass'), and its modal damping ratio ξ_1 .

f_1 - *Fundamental natural frequencies*

A floor can be regarded as a system of beams and plates spanning between columns. The portion of the floor between four columns is referred to as a 'floor bay' and it is usually taken as a reference for computing its natural frequencies. These may be computed using analytical closed-form expressions of simple structural elements with different boundary conditions. Some of the most useful formulas for this purpose have been summarized in Table 2.1.

Table 2.1 Fundamental modal properties of simple structures

Structure	Fundamental Frequency	Equivalent mass
	$f_1 = \frac{3.52}{2\pi} \sqrt{\frac{EI}{ml^4}} \quad (2.4)$	$M_1 = 0.64ml \quad (2.5)$
	$f_1 = \frac{\pi}{2} \sqrt{\frac{EI}{ml^4}} \quad (2.6)$	$M_1 = 0.5ml \quad (2.7)$
	$f_1 = \frac{22.4}{2\pi} \sqrt{\frac{EI}{ml^4}} \quad (2.8)$	$M_1 = 0.41ml \quad (2.9)$
	$f_1 = \frac{\alpha}{l^2} \sqrt{\frac{Eh^3}{12\mu(1-\nu^2)}} \quad (2.10)$ $\alpha = 1.57(1 + (l/b)^2)$	$M_1 = 0.25\mu lb \quad (2.11)$
	$f_1 = \frac{\alpha}{l^2} \sqrt{\frac{Eh^3}{12\mu(1-\nu^2)}} \quad (2.12)$ $\alpha = 1.57\sqrt{5.14 + 3.13(l/b)^2 + 5.14(l/b)^4}$	$M_1 = 0.17\mu lb \quad (2.13)$
	$f_1 = \frac{\pi}{2l^2} \sqrt{\frac{D_x + 2H\left(\frac{l^2}{b^2}\right) + D_y\left(\frac{l^4}{b^4}\right)}{\mu}} \quad (2.14)$	$M_1 \approx 0.3\mu lb \quad (2.15)$

where l and b are the length of the elements in the x and y directions, respectively; E and ν are the Young and Poisson modulus of the material, m and μ are the mass per unit of length and area, respectively; h is the thickness of the slabs, I is the moment of inertia of the beams, D_x and D_y are the bending stiffness per unit of length in the x and

y directions of the orthotropic slab, respectively; and H is a torsional stiffness parameter according to [43].

Additionally, there exists an energetic equivalence between the fundamental natural frequency of a system and its static deflection Δ under self-weight load, given by the following equation.

$$f_1 = \frac{\sqrt{g}}{2\pi} \frac{1}{\sqrt{\Delta}} \approx \frac{18}{\sqrt{\Delta}} \quad (2.16)$$

These kinds of expressions can be combined to obtain the fundamental natural frequencies of more complex systems using Dunkerley's formula. The final fundamental frequency of the system can be obtained by computing those of its composing elements and combining them as follows:

$$\frac{1}{f_1^2} = \sum_{i=1}^N \frac{1}{f_i^2} \quad (2.17)$$

In the example of Figure 2.12, the fundamental frequency of the bay can be computed from those of the girders, joists and the slab through calculating their static deflections under self-weight loads.

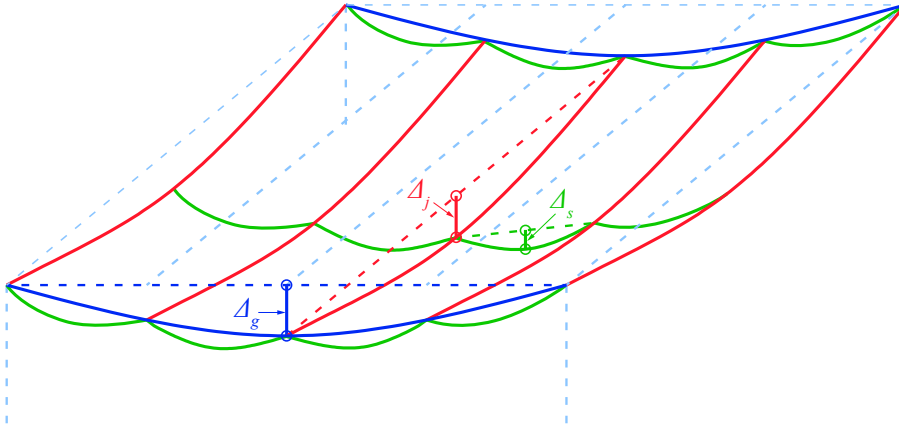


Figure 2.12: Example of Dunkerley's formula application on a framed floor

The presence of adjacent bays tends to increase the natural frequency values with respect to those computed for an isolated bay. The Concrete Center guideline [43] proposed a frequency amplification factor for single-spanning simply-supported floors depending on the frequency and span ratios between both adjacent bays. This enables to increase f_1 up

to a 50% in certain particular cases.

M_1 - *Equivalent mass of fundamental vibration mode*

This parameter represents the mass of an equivalent SDOF system that dynamically behaves as the contribution of the fundamental vibration mode on its maximum displacement point and can be computed as follows:

$$M_1 = \iint M(x, y) \phi_1(x, y)^2 dx dy \quad (2.18)$$

where $M(x, y)$ is the distribution function of the mass within the floor, and $\phi_1(x, y)$ is a function of the first mode shape.

The dynamic performance of a floor system is quite sensitive to this parameter (which is not easy to estimate accurately), as both resonant and impulsive responses are inversely proportional to it. If the floor structure is simple enough and there are guarantees that each bay behaves independently, simple approximations as those shown in Table 2.1 can be used. However, the additional contribution of adjacent floor bays to the final equivalent mass is an important concern, as single-panel or single-bay approximations of this parameter lead to high overestimations of the vibration level. In most of the design guidelines, this effect is accounted for by the computation of an effective area of the floor which 'participates' in the floor dynamic response, as follows:

$$M_1 = L_{eff} B_{eff} \mu \quad (2.19)$$

where B_{eff} and L_{eff} are the effective width (parallel to the 1st beams) and the effective length (parallel to the 2nd beams) of the floor as can be seen in Figure 2.14.

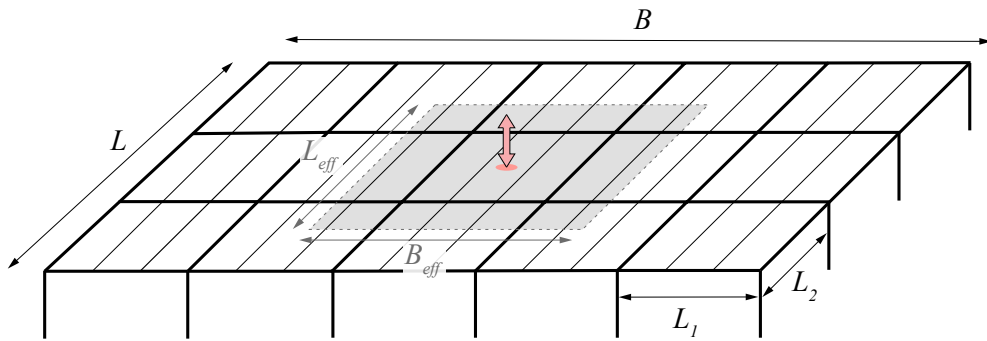


Figure 2.13: Effective floor area participating in the floor dynamic response

In their review paper, Middleton *et al.* provide a detailed comparison between expressions provided by different guidelines to compute L_{eff} and B_{eff} , highlighting that their origin is not clear, they lack consistency and they seem to be extracted from parametric FE analysis. Nevertheless, most of them seem to relate these magnitudes to the 4th root of a quotient between two stiffness parameters of the floor.

The Concrete Center design guide proposed to compute the effective mass with the formula of an orthotropic plate for a row of bays of dimension B and L_2 and then, increase this value depending on the dimensions of the adjacent row of bays.

An interesting study in this respect is the one performed by Willford *et al.* [44] in which the response of a floor is computed with different methods considering the presence of an increasing number of adjacent bays. Using FE models, the response decreases with the number of bays till stabilises at a certain level.

ξ_1 - Damping ratio

Damping is the term used to consider all the dissipation mechanisms that are involved in the dynamic response of a structural system and conceptually there can be classified within four types: i) viscous, ii) viscoelastic, iii) dry frictional, and iv) hysteretic. Many researchers have confirmed that the nature of these mechanisms is non-linear, mainly hysteretic and frictional, however, for low-amplitude vibrations, their contribution can be assumed to be linear viscous for mathematical convenience and without incurring significant inaccuracies.

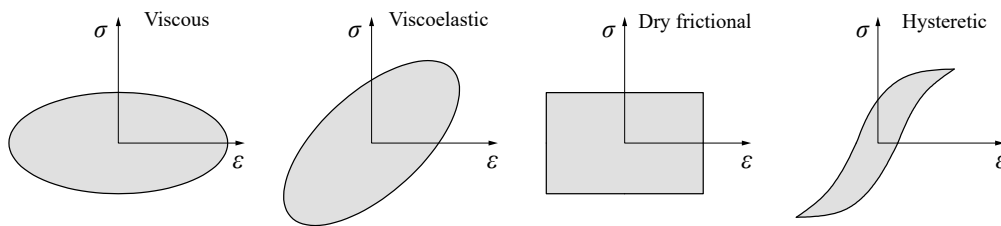


Figure 2.14: σ - ϵ hysteresis loops of the four different types of damping mechanisms

The main sources of damping within a floor are construction joints, structural connections, microscopic concrete cracks, false flooring or ceiling systems (when not rigidly connected to the structure), partitions and furniture. Thus, the final damping ratio is a summation of three contributions: i) the materials ii) the type of furniture and iii) the floor finishes. One of the most complete resources in this respect is the table 2.2 developed

by [47].

Damping has a great impact on the final resonant response of LFFs and also reduces (to a lesser extent) the rms impulsive response of HFFs. In fact, one of the main causes in modern floors is the lack of damping due to the removal of heavy furniture and partitions. This parameter can be increased by means of damping devices or vibration control strategies such as the one studied in this thesis.

Table 2.2 Damping ratio contributions for typical floor systems [47]

Type	Damping ratio [%]
Material damping	
Wood	6 %
Steel	1 %
Concrete	2 %
Composite	1 %
Furniture damping	
Traditional office 1-3 people with separation walls	2 %
Paperless office	0 %
Open plan office	1 %
Library	1 %
Houses	1 %
Schools	0 %
Gymnasiums	0 %
Finishes damping	
Ceiling under the floor	1 %
Free-floating floor	0 %
swimming screed	0 %

Simplified assessment of LFFs resonant response

The design guidelines for floor vibration available nowadays provide simplified methodologies for assessing the vibration of LFFs, all of which are similar, but differ in slight details. Here, the general framework for these methodologies is explained. Within them, the harmonic pedestrian load model described in Section 2.3.1 is used, and a resonant behaviour with one of the harmonics of the pedestrian load is assumed. Thus, the exciting harmonic force is equal to $\alpha_h Q$ (α_h should be taken from Figure 2.9 depending on the

guideline used). For the sake of simplicity, the pedestrian is assumed to be standing still at a given point on the floor. The peak resonant acceleration, $a_{p,res}$, is computed for the equivalent SDOF of the fundamental vibration mode. Then, the rms resonant acceleration, $a_{rms,res}$, is computed by dividing $a_{p,res}$ by $\sqrt{2}$. Finally, depending on the guideline chosen, some factors are included to consider additional aspects.

- $R_{cal,res}$: Applied by AISC DG 11. A reduction Factor for considering that the pedestrian is not always walking over the point of maximum nodal coordinate. This factor is equal to 0.5 for two-way mode shape structures and 0.7 for one-way mode shape ones. Parameter specifically calibrated for this guideline.
- ρ : Applied in SCI P354 and Concrete Center guidelines. A build-up factor is used to consider that the resonant response might not be fully developed due to the lack of corridor length.
- μ_e and μ_r : Applied in SCI P354 and in Concrete Center guidelines. The modal coordinates of the exciting and responsive points within the floor. They are used to consider that the exciting pedestrian and the human receiver of vibration might not be placed in the worst possible location on the floor.
- W_h : Applied in SCI P354 guideline. A weighting function to consider the perception of vibration by the human. This will be commented on later.
- K_{rm} : Applied on the Concrete Center guideline. An amplification factor of the response to consider the contribution of other vibration modes apart from the fundamental one.

A generalized expression for computing $a_{rms,res}$ would be:

$$a_{rms,res} = Q\alpha_h \left(\frac{1}{2\xi_1 M_1} \right) \left(\frac{1}{\sqrt{2}} \right) (R_{cal,res} \rho \mu_e \mu_r W_h K_{rm}). \quad (2.20)$$

Simplified assessment of HFFs impulsive response

For this, the impulsive model of the human walking load is used. First, the peak velocity after the footfall impact, $v_{p,imp}$, is computed by dividing $I_{eff}(f_p, f_1)$ by M_1 . Second, assuming an exponential free-decaying response with duration f_p , the rms velocity, $v_{rms,imp}$, is computed by integrating the time domain signal. Just as before, some additional factors are included depending on the guideline used:

- μ_e and μ_r : Applied in SCI P354 and Concrete Center guidelines.
- W_h : Applied in the SCI and Concrete Center guidelines.
- R_M : Applied in the AISC DG 11. A higher mode factor for considering the contribution of higher modes to the peak response of the floor.
- $R_{cal,imp}$: Applied in the ASIC DG 11. A calibration factor for the AISC DG method with a value of 1.3.

For values of ξ_1 lower than 0.1, the following expression can be used:

$$v_{rms,imp} = \sqrt{f_p \int_0^{(1/f_p)} \left(\frac{I_{eff}}{M_1} e^{-2\pi f_1 \xi_1 t} \sin(2\pi f_1 t) \right)^2 dt}, \quad (2.21)$$

$$v_{rms,imp} \approx \left(\frac{I_{eff}}{M_1} \right) \left(\frac{1}{\sqrt{2}} \sqrt{\frac{\left(1 - e^{-\frac{4\pi f_1 \xi_1}{f_p}} \right)}{\frac{4\pi f_1 \xi_1}{f_p}}} \right) (R_M \mu_e \mu_r W_h R_{cal,imp}) \quad for \quad \xi_1 < 0.1. \quad (2.22)$$

FE models of floors for VSLS

The VSLS can be also assessed by using an FE model of the floor, either to obtain a better estimation of its modal properties or to directly analyze its dynamic response on the FE software. FEA is especially interesting when the structural geometry is complex, as it provides more accurate and flexible solutions. Modelling a building floor is a complex task that requires skills based on experience, however, current design guidelines offer general advice in this respect. Here, some of these recommendations have been summarized:

Good practices in floor modelling for VSLS assessment

- Extension of the model
 - Floor motion, as seen in field measurements, is usually restricted to an extension of 4 or 5 bays. This is mainly due to two factors: i) The influence of frictional damping that causes energy loss over the distance, and ii) in real buildings, the mass and the frame scheme are different from bay to bay

and thus, the vibration modes do not involve the motion of large floor areas. Because of this, the use of large floor models with uniform framing and mass distribution should be avoided as they will over-predict the mass in motion. A good practice is to model the excited bay and the adjacent ones i.e. a 3 x 3 grid of bays.

- Slab definition
 - Use of orthotropic shell or plate 2D elements for the slab. If this is not possible
 - Increase a 35% the Young modulus of the concrete, as in this regime of vibration amplitude concrete is stiffer.
 - Consider the concrete as uncracked.
 - The use of elements of 1/10 of the bay size is usually adequate. Use from 3 to 4 elements between secondary beams.
- Framing members
 - Use regular three-dimensional beam elements with a mesh compatible with the one used for the slab.
 - Down-stand concrete ribs or steel members must be modelled explicitly with their individual section properties and including their offset with respect to the slab centre line.
 - Do not restrain their lateral movement
 - Do not model the columns with pinned or fixed supports, Natural frequencies are quite sensitive to this parameter. Model the columns extending to the stories below and above.
- Connections
 - All the beam-beam and beam-column connections must be considered as fixed and not hinged ones. Their rotational stiffness in the VSLs amplitude range is much higher than the one used for static calculations. Only connections of steel truss joists might be considered as hinged.
 - The same applies to the longitudinal shear connection between steel beams and concrete slabs which must be assumed to be rigid.

- Partitions
 - Facades can be usually well represented through restraining the vertical displacement of the edge beams.
 - Non-Structural partitions may be modelled with shell elements or rows of spring elements of around 1200 kN/m².
- Mass
 - Consider the frequent dead load as well as heavy furniture or machinery elements present on the floor.
 - Consider, if likely to be there, a 10% of the design live load as a distributed mass on the floor for the VSLS assessment.

VSLS assessment of floors by means of FEA

There are different alternatives in order to using a FE model to assess the VSLS of a floor. Here some of them are summarized:

- *Modal analysis*: It enables the extraction of the natural frequencies and vibration modes of the floor.
- *Harmonic and spectral analysis*: It is useful to rapidly estimate the dynamic resonant response of LFFs, as a harmonic loading model is used, however, it does not enable considering non-linearities or moving loads along walking paths.
- *Transient analysis*: A time integration of the equations of motion of the floor is performed (which allows considering the presence of walking paths and moving loads). This can be done in two ways: i) assuming modal superposition or that the dynamic response of the floor is a linear combination of the modal responses of each vibration mode, which results quite convenient in terms of computational cost, but prevents considering non-linearities, and ii) perform a direct integration of the equations of motion, which involves a great computational cost but allows considering non-linear effects. Transient analysis may be used to analyze both LFFs and HFFs response by using the harmonic loading model described in equation 2.1.

2.3.3 The vibration receiver: vibration limits

In VSLs problems the vibration receiver is represented by a point or points on the structure in which the vibration needs to be limited due to two main reasons: i) the comfort of users present on the structure, or ii) the sensitivity requirements of precise equipment used in laboratories or hospital facilities (usually two orders of magnitude more restrictive than human sensitivity). This doctoral thesis is more related to the first reason, for that, the human vibration receiver is the only one explained in detail within this document.

The vibration receiver is usually defined by a limit on the amount of vibration experienced by these critical floor locations. Usually, the VSLs problems of LLFs are related to human comfort, whereas HFFs are devoted to spaces allocating precision machinery.

Vibration Limits for human comfort

The perception of vibrations by humans has been covered in detail by Pavic and Reynolds [48] and Živanović *et al.* [20].

Generally, the effects of vibration perceived by humans are classified into five groups: i) degraded comfort, ii) interference with activities, iii) impaired health, iv) occurrence of motion sickness, and v) perception of low-magnitude vibration. The first four groups of effects are usually associated with vibrations occurring in transportation or mechanical and industrial ambiances, however, floor vibrations are closely related to the fifth one, as regular complaints in residential or office buildings are related to low-amplitude motion that in other ambiances would not be problematic. In this context, the acceptability of floor vibrations is governed by the concept of 'vibration perception threshold'. This threshold is defined as the minimum vibration perceivable by humans. In order to quantify the vibration a relevant dynamic motion descriptor has to be chosen. Typically acceleration has been chosen for the ease of instrumentation.

Initially, researchers studied the magnitude of this perception threshold providing different values and perception curves with data obtained in laboratory conditions. Some of the most relevant early works were developed by Reiher *et al.* [49], Goldman [50], and Wright *et al.* [51]. Later, after the development of the ISO 2631 [27] and the compendium works carried out by Irwin [52] and Griffin [53] it was demonstrated that human response to vibration, although fairly unpredictable, follows certain patterns mainly dependent on the frequency content of the vibration, the position of the human body, and the situation.

In fact, it was observed that vibration with frequency content between 4 to 8 Hz, acts as a whole body 'base' excitation causing resonances with the internal organs, which makes humans more sensitive to it. This knowledge is well summarized into three elements:

- *Weighting curves* as those provided by ISO 2631 and depicted in Figure 2.15 that consider the human sensitivity to vibration depending on its frequency and position. They are filters that enable transforming a 'rough' vibration into a 'perceived' one.

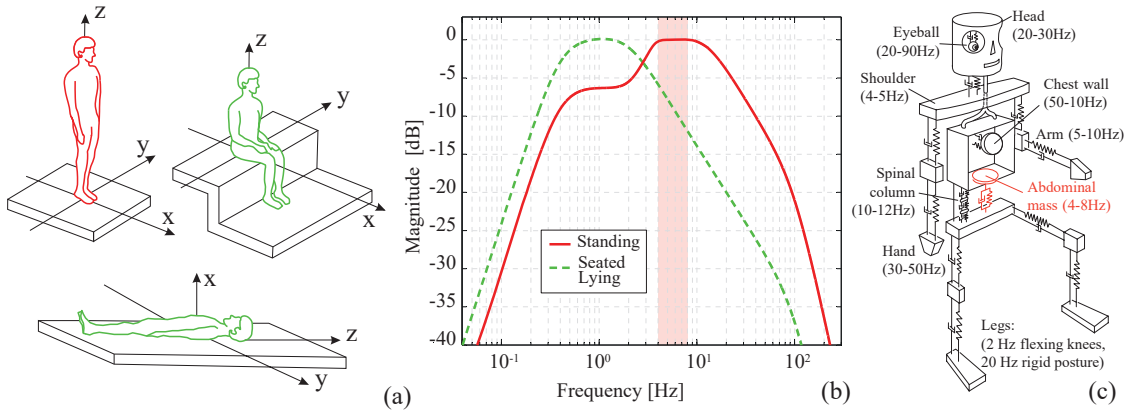


Figure 2.15: ISO 2631 Weighting functions [27]. (a) Human body positions. (b) Weighting functions. (c) Human mechanical model [54]

- *Baseline perception vibration values.* These values have been discussed within the literature. Initially, several authors proposed inconsistent values of minimum noticeable vibration (0.003 m/s^2 [55], 0.1 m/s^2 [56], or 0.04 m/s^2 [57]). Then, Jones and Saunders proposed 0.01 m/s^2 of peak acceleration as a better contrasted 'feel'-'no feel' limit. Finally, the standards ISO 10137 and BS 6472 established this limit in a weighted rms acceleration limit between 4 to 8 Hz of 0.005 m/s^2 which up to date, has been widely accepted in the VSLs field.
- *Multiplying factors.* The maximum admissible continuous weighted acceleration on a floor is usually defined with a multiple of the baseline value previously defined (0.005 m/s^2) [52]. These factors have ended up being called 'response factors' here denoted with R . Current design guides and standards provide different values of R mainly depending on the function of the floor to be assessed (residential, office, hospital...). Table 2.3 compares the different factors provided by different sources. The reference documents for this are: the standard ISO 10137 [19], the BS 6472 [59], the existing design guides for floor vibration ([23], [24], [43] and [26]) and for hospital facilities the document HTM 08-01 of the UK health department [60]. It

is important to note that the factors provided by the HIVoSS guideline are much more permissive than the rest of the sources, which results in controversy within the engineering community.

Table 2.3 Comparison of response factors values for different floor functions depending on the guideline used [19], [23], [24], [26]

Floor use	Time	ISO10137	AISC DG 11 or SCI P354	HIVoSS
Critical Workspace	Day	1	1	1
	Night	1	1	1
Health - Hospitals	Day	2 to 4	2	2 to 8
	Night	1.4	2	2 to 8
Residential - Hotel	Day	2 to 4	8	8 to 32
	Night	1.4	8	8 to 32
Quiet Office - Open plan	Day	2	8	8 to 32
	Night	2	8	8 to 32
General Office	Day	4	8	8 to 32
	Night	4	8	8 to 32
Schools - Education	Day	4	8	2 to 8
	Night	4	8	2 to 8
Shopping malls	Day	-	4 to 22	8 to 32
	Night	-	4 to 22	8 to 32
Workshops	Day	8	8	32 to 128
	Night	8	8	32 to 128
Sports - gymnasiums	Day	-	-	32 to 128
	Night	-	-	32 to 128

Before closing this topic, it is important to comment on the concept of 'vibration criteria' (VC), the parameters used for assessing the amount of vibration. Today, two main ones are used: i) the rms weighted acceleration (above mentioned), and ii) the vibration dose value (VDV).

- *rms weighted acceleration*: This VC is usually used to assess continuous periodic vibrations. The aforementioned perception limit of 0.005 m/s^2 is, in reality, a rms weighted acceleration. This means that the acceleration is filtered through the

weighting functions depicted in figure 2.15 and then, a rms acceleration value is computed as follows:

$$a_{w,rms} = \sqrt{\int_{t_1}^{t_2} \frac{a_w^2(t)}{t_2 - t_1} dt}, \quad (2.23)$$

The only inaccuracy of using the $a_{w,rms}$ as a VC is the definition of the integration time $t_2 - t_1$. Many authors suggest using from 1 s to 60 s, however, Eriksson [61] proposes 10 s as more appropriate for assessing floor vibrations, thus developing the function $a_{w,rms,10}(t)$. The maximum value of a rms function $a_{w,rms}(t)$ is called the 'maximum transient vibration value' (MTVV) and often is used as a VC for comparison with the vibration limits aforementioned. Figure 2.16 provides a graphical comparison of all these vibration parameters.

- *VDV*: is a cumulative measure of the vibration transmitted to a human receiver during a certain period of time $t_2 - t_1$ (as can be seen in Figure 2.16 the VDV increases monotonically as vibration accumulates). The VDV was first proposed in the UK standards [59] as a parameter that enabled the assessment of any type of vibratory motion (periodic, transient...), and it is computed as follows:

$$VDV = \left[\int_{t_1}^{t_2} a_w^4(t) dt \right]^{1/4}. \quad (2.24)$$

The BS 6472 has proposed certain vibration limitations applicable to floor vibration assessment in terms of the VDV, however, they seem to be more applicable at an experimental quality assessment of already built floors, than at a design stage.

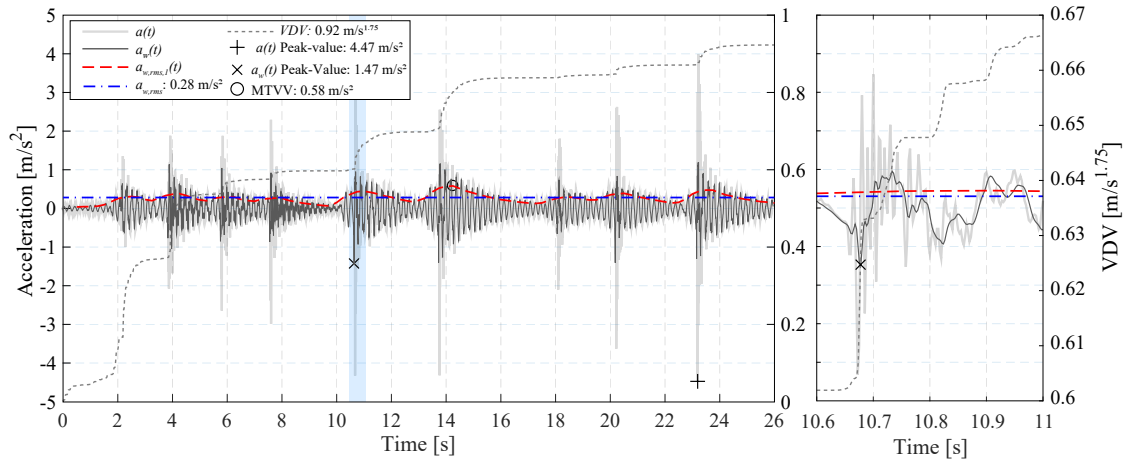


Figure 2.16: Comparison between different VC to be used to assess the amount of vibration perceived on a structure

2.3.4 Standards, design guidelines and codes for VSLS of floors

The VSLS of floors is a topic covered by different standards, design guidelines and codes. This subsection provides an organized summary of these three types of sources that have been regularly referenced during the previous sections.

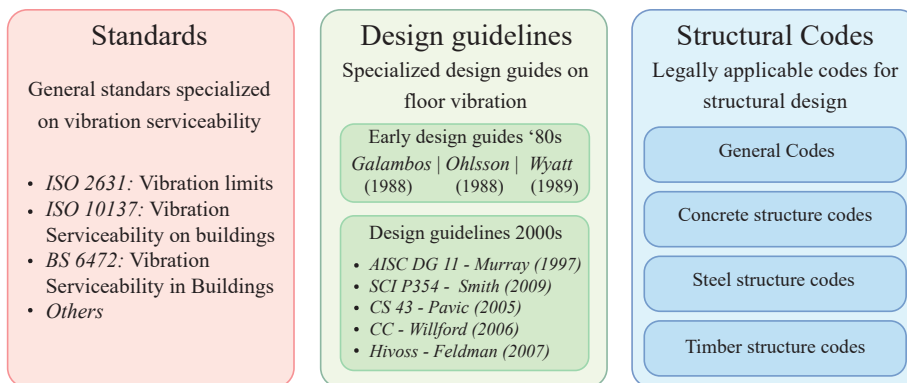


Figure 2.17: Standards, design guidelines and codes reviewed in the present section

Standards

Today, there are three main standards dealing with VSLS problems in structures:

- *ISO 2631: Evaluation of human exposure to whole-body vibration - Part 1: General requirements* (1985) [27]. It is focused on the human vibration receiver. It provides

the weighting functions above mentioned and vibration ranges related to human comfort and vibration perception.

- *ISO 10137: Bases for design of structures - Serviceability of buildings and walkways against vibrations* (1992) [19]. This is a holistic and general document related to VSLS issues in buildings. The information presented in the document annexes is especially useful as it provides human loading models and vibration limitations.
- *BS 6472: Guide to evaluation of human exposure to vibration in buildings. Vibration sources other than blasting* (1984) [59]. It is similar to ISO 2631, and it is more focused on the vibration measurement to assess the adequate dynamic performance of a building. It made a significant contribution to the VSLS field by providing the VDV as a new VC.
- Other Standards (with reduced influence) that might be used to assess the human perception of vibrations are i) the German one DIN 4150-2 (with an outdated procedure based on velocity) [62], ii) the Italians UNI 9614 [63] and UNI 9916 [64], or iii) the American one ANSI S3.18 [27].

Design Guidelines

In order to contextualize the Design Guidelines currently available, the history of the VSLS of floors has been studied, dividing it into three chronological periods:

1. *'The initial research and early guidelines'*: Especially during the '60s, '70s and '80s, many researchers addressed the topic of floor vibration, providing new significant knowledge related to human loading models, vibration perception limits and VSLS assessment methods. Some of the most relevant works which provided a general framework for its assessment are those conducted by Lenzen [65], Galambos [66] and Murray [67], [68] in the U.S.A. (The majority of them published on a report series called of *Studies in Engineering Mechanics at the Center for Research in Engineering Science of the University of Kansas* on report numbers 1, 3, 4, 5, 9, 12, 16, 29, 30 and 32), Allen and Rainer [69], [37] in Canada, and Ohlsson [36] in Europe. A complete review of most of these works was developed by Ellingwood *et al.* [70].

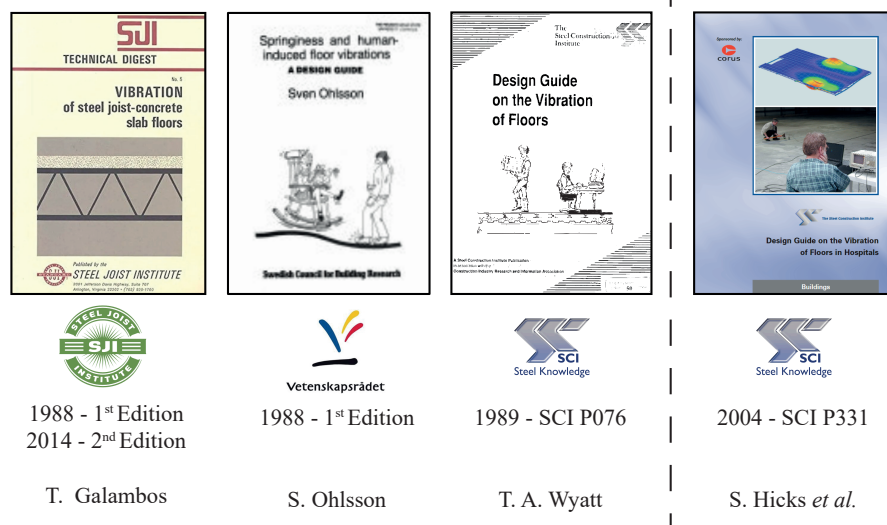


Figure 2.18: First generation of early design guidelines for floor vibration. References from left to right in the image [71], [72], [73], [74]

This first period ended with the publication of ISO 10137 and the development at the end of the '80s and beginning of the '90s of a first generation of design guidelines for floor vibration: i) The one published by the Steel Joist Institute in 1988 in U.S.A. and led by Galambos [71] (forerunner of today's AISC DG 11). ii) The one published in 1988 by the Swedish Council for Building Research led by Ohlsson [72], and iii) the one published by the Steel Construction Institute in the U.K. in 1989 and led by Wyatt [73]. These documents mainly provided simplified methodologies to analytically assess the VSLs of floors using a primitive characterization of the human dynamic loading.

2. *'The design guidelines'*: this covers from 1990 to 2007 or 2009 and can be seen as a consolidation period in which the concepts and methodologies previously proposed were improved and refined. Additionally, this phase coincided in time with the Millennium Bridge lateral vibration event, which impuled the research associated with the HIVs. This explains that the main researchers who led this consolidation period were based in the U.K. as, for example: Pavic ([48], [75], [76], [77], [78]) and Brownjohn ([79], [80], [81]) in the University of Sheffield, Willford in the engineering consultancy company ARUP ([44], [45], [42], [82], [83], [84]), Hicks in the Steel Construction Institute ([85], [86]) who in 2004 led the redaction of the *Design guide on the vibration of hospital floors SCI P331* (Continuation of SCI P076 and forerunner or SCI P354) [74], or Ellis ([39], [87], [88]) in the Building Research

Establishment (BRE).

This period is characterized by the publication of 5 widely accepted design guidelines for floor vibration (those depicted in Figure 2.19):

- *AISC DG 11 - Vibrations of Steel-Framed Structural Systems Due to Human Activity* [23]: it was impulsed by T. Murray and published by the AISC based on the previous design guide by the SJI [71]. It is mainly focused on Steel-framed floors based on steel beams or joists that support a concrete slab. The 1st edition of 1997 accounted for a calibrated method to predict the dynamic behaviour of these floors, whereas the 2nd one of 2016 provides additional guidance on the FE modelling of these floors.
 - *SCI P354 - Design of Floors for Vibration: A New Approach* [24]: it is an updated version of SCI P076 and SCI P331 and it was released in 2009. It was impulsed by A. Smith and published by the SCI. The methodology proposed in it is valid for any kind of floor system.
 - *Appendix G of the Post-tensioned Concrete Floors Design Handbook* [25]: which provides a brief Appendix on how to deal with VLS in this type of floor. It was impulsed by A. Pavic and published in 2005 by the Concrete Society.
 - *A Design Guide for Footfall Induced Vibration of Structures* [43]: this guideline is not specific for floors, as it also covers footbridge vibrations. The Simplified methodologies proposed do not seem to be calibrated as those of the AISC and SCI Guides. Additionally, it provides a clear summary of how to develop FE models of floors. It was published in 2006 by the Concrete Center and impulsed by M. Willford.
 - *Hivoss: Vibration Design of Floors - Guideline* [26]: this document was developed as a result of a European project financed by the Research Fund for Coal and Steel which involved a group of experts coming from different institutions (universities, research centres and companies). The final document, published in 2007, provides a methodology to predict floor vibrations that makes use of a velocity-based VC and establishes vibration limits substantially less restrictive.
3. *'The technical sophistication'*: this is the third and last period from 2010 to today and will be explained in detail in the section 2.3.5.



Figure 2.19: Design guidelines for floor vibration [54], [23], [25], [43] and [26]

Codes

Generally, the references to the VSLs of floors in the legally applicable codes for building design are scarce and fuzzy. The majority of the codes just provide brief recommendations on limits applicable to the fundamental natural frequency of the floor (see Table 2.4). Some of the most relevant standards are listed below:

- General codes for structural design:
 - Europe - *EN 1990:2002, Eurocode 0* [89]: Within its normative Annex 1 clause A.1.4.2 it states that for buildings “*The serviceability criteria should be specified for each project and agreed with the client.*” or they may be defined in the National Annex (NA). This code provides clause A.1.4.4 completely devoted to the VSLs of buildings in which general indications for analyzing it are given and movements of people are recognized to be critical dynamic actions on buildings. Additionally, this clause references the norms EN 1991-1-1 and ISO 10137 for further guidance. Finally, clause A.2.4.3 covers the VSLs of footbridges due to pedestrian traffic, stating comfort limits in terms of acceleration.
 - Europe - *EN 1991-1-1:2002, Eurocode 1* [90]: In clause 2.2 devoted to imposed loads on buildings, it is stated that “*If resonance effects from synchronised rhythmical movement of people or dancing or jumping may be expected, the load model should be determined for special dynamic analysis. NOTE: The procedure to be used may be given in the National Annex.*” In this respect,

the NA of the U.K. establishes frequency limits for regular floors and those subjected to human rhythmic actions (which are the same given in the British code BS 6399 [91]. See Table 2.4) and makes reference to the document BRE Digest 426 for further guidance [92].

- Canada - *National Building Code of Canada NBC:2020*: Clause 4.1.3.6 (applicable to general buildings) is devoted to VSLS of floors and states that floors should be designed so that any adverse comment arises due to vibration. Moreover, if rhythmic activities are going to be held on the floor and its natural frequency is less than 6 Hz, the effects of resonance should be investigated. The notes to this clause refer to the document *Structural Commentaries (User's Guide - NBC 2015: Part 4 Division B)* [93] for further guidance on the VSLS assessment. This document refers to [24], [23] and [94] for floor vibration under walking loading, and provides a methodology for the assessment of vibration resonance due to rhythmic actions (based on the research developed by Allen in the '80s and the '90s, [95], [96] and [97]). Finally, in clause 9.23.4.2 (devoted to the design of housing or small buildings) the vibration is considered with a concentrated load deflection criterion that modifies the 'design span tables' used for sizing lightweight joists and beams made of steel and timber.
- U.K. - *BS 6399-1:1984, British Standard* [91]: It establishes frequency limits for floors and provides the normative Annex A for modelling dancing and jumping actions on floors (Table 2.4).
- Spain - *CTE DB-SE, Código Técnico de la Edificación* [98]: This code provides general rules applicable to building design. In terms of VSLS it establishes limits in the fundamental frequency of the floor depending on its function (Table 2.4).
- Italy - *NTC:2018, Norme Tecniche per le Costruzioni* [99]: makes reference to the standards UNI 9614 [63] and UNI 9916 [64] for checking the VSLS. Furthermore, it provides frequency limits to be imposed on floors depending on their function, which could be overcome by performing a detailed vibration analysis (Table 2.4).
- Concrete structure codes do not usually include specific mentions of VSLS of floors, for example, in Eurocode 2 or EN 1992-1-1:2004 [100] it is established that: *"Other limit states (such as vibration) may be of importance in particular structures but are not covered in this standard"*. The U.S.A. code ACI 318-19 [101] does not consider

the VSLs, nevertheless, its commented 2019 version includes a comment to clause 24.1 in which it is stated that VSLs is not generally an issue on reinforced concrete floors, but might be considered in long-span floors with open-plan areas, in floors devoted to precision manufacturing or laboratory spaces or in facilities subjected to rhythmic activities. For its assessment, it refers to the following documents [43] and [102].

- Steel structure codes often provide certain guidance in this respect (as steel floors are more prone to vibrate than concrete ones due to their lighter nature). Some of the most relevant are:
 - Europe - *EN-1993-1-1:2005, Eurocode 3* [103]: Its clause 7.2.3 refers to Eurocode 0. Additionally, the code allows the definition of VSLs limits for buildings in the NA. In this respect, countries such as Spain, Cyprus or Italy, recommend floor frequency limits (see Table 2.4), others such as Sweden reference specific literature [104] and the rest, such as Germany or the U.K. remain indifferent.
 - Spain - *CTE DB-SE-A: Código Técnico de la Edificación* [105]: Clause 7.2.1 states that excessive HIVs tend to occur in lightweight and long-span floors. Then, clause 7.2.2 provides a simplified methodology to check the VSLs due to HIVs based on the acceleration response to a drop. The maximum admissible acceleration is a function of ξ_1 and f_1 , and the values proposed for ξ_1 are much higher than those generally used for steel floors.
 - India - *IS 800:2007, Indian Standards* [106]: This guideline provides a relatively detailed 'Annex C' to the clauses 5.2.2.2(b) and 5.6.2. It provides general frequency limits (Table 2.4) and simplified methodologies for computing the dynamic response of floors under a drop.
 - U.S.A. - *AISC 360-10, American Institute of Steel Construction* [107]: This code recognises the importance of VSLs in the design of steel-framed floors, however, it does not provide any vibration limit. Instead, it refers to the 1st edition of the AISC DG 11 [23] for that purpose.
- Timber Structures codes usually cover in more detail the floor VSLs issue. Their methodologies typically try to prevent excessive impulsive vibrations of lightweight HFFs.

- Europe - *EN-1995-1-1:2004, Eurocode 5* [108]: Clause 7.3 of the code is dedicated to VSLS, but the provisions it contains are outdated (proof of this is that the clause recommends assuming a value of $\xi_1 = 0.01$ which today is known to be quite low for a timber floor). In Clause 7.3.3 the code provides a simple but obsolete methodology to assess the VSLS of residential floors with f_1 higher than 8 Hz (based on the early design guide by Ohlsson [72]) that uses an impulsive peak velocity and a concentrated load deflection under a 1 kN load, d_1 as VCs for the floor.

The new draft of this code includes a modified methodology (based on the one proposed by Abeysekera *et al.* [109]) in which the VSLS assessment is divided between LFFs and HFFs (in a way similar to the one explained in Section 2.3.2) and using the response factor R as a first VC, however, the concentrated load deflection d_1 will remain as a second VC. According to Abeysekera *et al.*, this last VC has been preserved to ensure serviceable designs for lightweight floors (those with a mass less than 10 times the walker mass), as for them the dynamic loading models currently available are no longer valid. In this sense, ” [...] *it makes sense to keep the point load deflection limit in place for these floors. The stiffness criterion allows for an approximate comparison of the proposed performance levels with existing national requirements. However further work is needed to ensure the stiffness criterion and the response factor correspond.*”

Table 2.4 Frequency limits belonging to different standards, design guidelines and design codes, applicable to floors' fundamental natural frequency and related to their VSLs assessment. References from top to bottom: [91], [110], [93], [98], [99], [103], [106], [108], [109], [23], [24], [43], [19], [59].

Code	Type	Frequency limits
BS 6399	U.K.	$f_1 > 8.4$ Hz for rhythmic loading $f_1 > 4.0$ Hz regular floors
NBC: 2000	Canada	$f_1 > 6$ Hz for rhythmic loading or detailed analysis
Comments NBC: 2000	Canada	Minimum frequency depending on floor use and Live Load used for the design
CTE DB-SE	Spain	$f_1 > 8.0$ Hz for gymnasiums and sport centers $f_1 > 7.0$ Hz Party rooms and public places without seats $f_1 > 3.4$ Hz Entertainment venues with seats
NTC:2018	Italy	$f_1 > 5.0$ Hz for rhythmic loading $f_1 > 3.0$ Hz regular floors
EN 1993-1-1	Cyprus	$f_1 > 9.0$ Hz for rhythmic loading on steel floors $f_1 > 5.0$ Hz regular steel floors
IS 800:2007	India	$f_1 > 8.0$ Hz for rhythmic loading on steel floors $f_1 > 5.0$ Hz regular Steel floors
EN 1995-1-1	Europe	8.0 Hz for regular timber floors
New EN 1995-1-1	Europe	$f_1 > 4.5$ Hz for regular timber floors or special analysis is needed 8 Hz limit between LFFs and HFFs
AISC DG 11	U.S.A.	9 Hz limit between LFFs and HFFs $f_1 > 15.0$ Hz no complaints registered in composite floors
SCI P354	U.K.	f_1 should be higher than 3.0 Hz generally 8.4 Hz limit between LFFs and HFFs $f_1 > 24$ Hz floor unresponsive to HIVs
CC Guide	U.K.	10 Hz limit between LFFs and HFFs
ISO 10137	International	8-10 Hz fuzzy limit between LFFs and HFFs
BS 6472	U.K.	7-10 Hz fuzzy limit between LFFs and HFFs

2.3.5 Latest developments on VLS of floors

During the past decade, the research field related to VLS of floors has gone through a period of sophistication. In this context, the researchers' interest has been focused on main challenges:

The improvement of the current dynamic human loading models

Most of this research is a continuation of the works begun by Pavic and Brownjohn in the late '90s. Two key contributions that reviewed the current state-of-the-art and stated some of the future challenges to better characterize the human dynamic loading are those developed by Muhammad *et al.* [111] and [112]. Looking in more detail, some researchers like Varela *et al.* [32], Ahmadi *et al.* [113] and Shahabpoor *et al.* [114] have been focused on enhancing the experimental characterization of the human walking load by employing more sophisticated measurement techniques such as instrumented suits and in-shoe sensors. Moreover, continuing with the work initiated by Živanović *et al.* [29], intensive research has been done on developing new probabilistic and stochastic dynamic human loading models like those proposed by Chen *et al.* [115][116], Peters *et al.* [33], Brownjohn *et al.* [117] or García-Diéguez *et al.* [118].

The Human-Structure Interaction (HSI) on floors

HSI research field formally started in 1997 with the work of Ellis and Ji [119] that aimed to characterize the influence of non-moving spectators on the dynamic response of stadium grandstands (changing their empty natural frequencies and increasing substantially the modal damping). Despite this, similar changes in the modal parameters of lightweight timber and composite floors had already been described by Lenzen [65], Ohlsson [36] and Ebrahimpour *et al.* [120] a decade before (in fact, in 1999 the ATC Design 1 guide developed by Allen and Murray [94] recommended the use of viscous damping ratios of up to 12% when designing heavily populated floors). These first studies related to HSI are well reviewed by Sachse *et al.* [121]. Then, extensive research was developed to better characterize this interaction for moving pedestrians [122], and several models were studied to reproduce it (of which probably the simplest one replicates the human as a moving SDOF system attached to the structure and accompanied by a load). Most of this work (the majority related to footbridges' vibration) has been collected and organized by Shahabpoor *et al.* in a review contribution, [123]. As floors are increasingly becoming more lightweight the HSI is starting to be a relevant phenomenon to consider when assessing

their VSLs. Consequently, in recent years some works have emerged on this line. For example, Gaspar *et al.* [124] considered the HSI influence applied for jumping activities on floors, demonstrating that conventional force models over-predict the floor dynamic response by more than 100%. Liu *et al.* [125] applied the interactive model first developed by [126] and studied its influence on the response of composite floors. In 2020, Zhu *et al.* [127] published the first experimental contribution explicitly devoted to a case of study in which the influence of HSI on the dynamic response of a large cantilever floor was assessed. Acceleration reductions of about 8% to 20% on the peak response of the floor were quantified due to the influence of HSI. A year later, a more general work performed by Mohammed *et al.* [128], demonstrated a 20% reduction on the FRF peak of a floor due to the presence of six walking humans, with respect to the empty situation. To conclude, more recently, Shahabpoor *et al.* [129] has satisfactorily performed the calibration of a 2-DOF HSI model on a floor laboratory structure.

The revision and enhancement of VSLs assessment methodologies

Once the design guidelines depicted in Figure 2.19 have been consolidated within the engineering community, some interesting works have emerged comparing the accuracy of the different methods these documents propose. For example, Muhammad *et al.* [130] have performed a detailed comparison of all these methods on three composite floors. For each guideline, an FE assessment and a simplified one have been carried out and their results have been contrasted with the actual vibration value measured on the floors. They conclude that the AISC DG 11 is the one which performs better while Hivoss "*appears to be an outlier and highly inaccurate*". In the same line, Royvaran *et al.* [131] have contrasted the simplified methodologies of four design guides against a database of 50 composite steel floors collected by Murray (that classified them as satisfactory or unsatisfactory). Again the AISC DG 11 performed the best while Hivoss "*provided a poor prediction accuracy*". Of particular interest is the review work developed by Gonçalves *et al.* [132] focused on the development of more realistic VSLs assessment methods for floors. In their conclusions they state five main challenges for the future: i) the use of probabilistic loading models for moving pedestrians, instead of static ones, ii) the integration of the HSI into the assessment methods (points i) and ii) have already been commented), iii) the better characterization of human activity on floors with movement tracking techniques (a key contribution in this respect is the one carried out by Abdeljaber *et al.* [133] related to the development of a video-vibration monitoring system to identify walking patterns in an office), iv) further investigate the effect of non-structural elements (as Devin *et al.* [134] have done in 2019 with a review paper), and v) the generalize use of probabilistic vibration-dose-based VC

(as has been researched by Setareh [135] by comparing the VDV against other VCs on an office floor). Additionally, relevant recent works difficult to classify are: i) the one done by Chen *et al.* [136] (which investigates the transmissivity of vibrations from an active floor of a building, being loaded, to a passive one) ii) and the one of Caballero-Garatachea *et al.* [137] (one of the few existing works related to vibrations of reinforced concrete-joint floors, widely used in Mediterranean and Latin American countries).

The advance on the VSLS of timber floors

In the last decade, due to the rise of EWPs and their increasing use in the timber construction industry, researchers have turned their attention to reviewing and further studying the dynamic performance of timber floors.

Timber floor vibrations started to be deeply investigated in the early '60s and '70s when a high number of complaints from wooden building occupants related to vibration issues arose. Some of these early surveys related to VSLS of timber floors were developed by Russel [138] in the U.S.A., Hansen [139] in Norway, or Onysko [140] who was specially prolific in Canada. A contribution of Whale reviews this initial period of the timber vibration research field. Until that time, traditional proportion-sizing rules based on deflection tests used by designers (as those proposed in 1820 by Tredgold *et al.* [142] based on deflection tests under 750 lb punctual loads limited to $L/480$) have produced serviceable floors in terms of vibration. Hence, as most of these issues occurred on new floors designed with pioneer 'working-stress' principles, many authors (for example, Chui *et al.* [143]) concluded that new floors were more flexible due to the refinement of design methods, use of stiffer materials and new construction techniques.

From this early stage and onwards, many competing and divergent design criteria for timber floors emerged to prevent structural designers from incurring into lively designs. In 2000 Hu *et al.* [144] wrote a review paper classifying these criteria into six groups, depending on the chosen parameters to be limited. However, this document prefers to conceptually divide them into two groups, those based on i) empirical rules or ii) simplified engineering mechanics principles.

- *Empirical rules:* these are mainly based on floor testing databases and tried to develop inequations representing a frontier between acceptable and non-acceptable tested floors. These expressions depended on some mechanical parameters of the floor as, for example, its fundamental natural frequency f_1 , a static deflection to a

1 kN punctual load d_1 , a static deflection over a distributed load d_d , or its span L_1 . Some of the first design criteria of this type were proposed by: i) Reiher *et al.* [49] based on f_1 and d_1 and tested on steel and wooden joists with less than 24 ft, ii) the US Federal Housing Administration (FHA) which limited d_d to $L/360$ claiming that: ‘*The design deflections set forth herein have been established on the basis of reducing vibration of floors [...]*’ iii) Russel [138] and Hansen [139] for timber floors and only dependent on d_1 , iv) Onysko [140] depending on L_1 and d_1 or, v) Dolan *et al.* [145] that proposed a minimum f_1 of 15 Hz for bare timber floors. Later, Hu *et al.* [146] [147] based on these principles, developed some famous expressions in the early 2000s that related d_1 and f_1 for different types of timber floors. These rules have inspired some of the design criteria currently present in some standards or guidelines, where these rules have been transformed into simplified maximum span or deflection limitations: i) the Canadian CLT Handbook, ii) the Design provisions for vibration serviceability of joisted floors of the Canadian code, or ii) the Canadian Standard CSA-086 in its Appendix A.8.5.3. These kind of criteria are useful when designing floors similar to those used to calibrate the empirical rule. A relevant review paper of these formulations is the one of Kalkert *et al.*

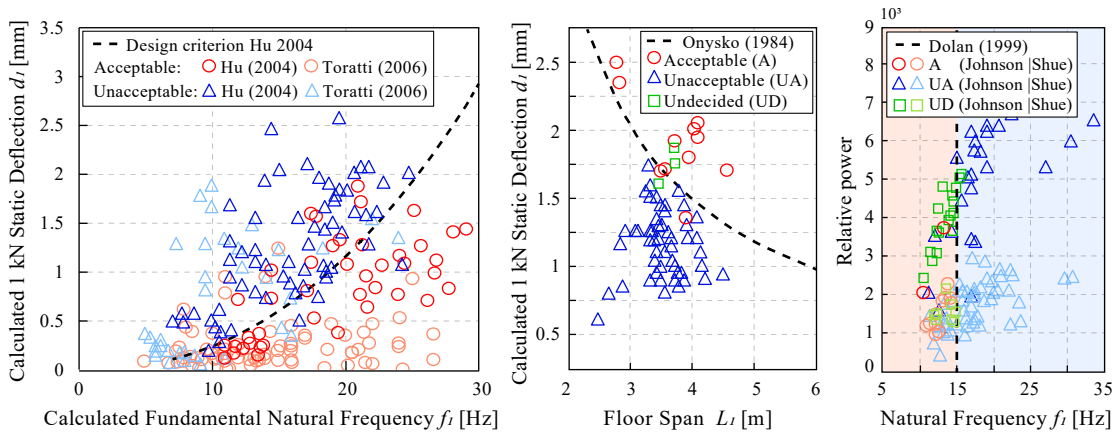


Figure 2.20: Empirical Design Criteria Proposed for timber floors

It is important to consider that the majority of these formulations have been derived to serve as a tool for designers, thus, many authors proposed design rules based on ‘calculated’ and not ‘experimentally measured’ parameters. Because of this, the development of these rules was in most cases accompanied by a detailed assessment of the accuracy of the methods used to compute these parameters. Of special importance for the authors was to study the influence of the transversal floor stiffness on d_1 and on its dynamic response. In fact, it is widely accepted in the timber

engineering community that having a proper bracing system between joists is crucial to minimize vibrations [149].

- *Simplified Engineering mechanics formulations:* in these formulations, a predicted system response to a disturbing force is compared to a limiting state condition (usually a dynamic response, acceleration or velocity is computed and compared to a limiting value). These kinds of design criteria are more similar to those previously reviewed for concrete and steel floors which were investigated by Lenzen and Murray [67] in U.S.A or Allen *et al.* [69]. In Europe, these methods applied to timber floors were mainly researched by Ohlsson [72] in Sweden and by Smith *et al.* [149] in the U.K. on a Research project impulsed by the Timber Research and Development Association (TRADA) trying to assess the VSLs of timber floor with their response to a heel-drop. The first version of Eurocode 5 was an early version of these criteria trends which today seem to have prevailed in modern codes and guidelines. In fact, the new Draft of EC 5 will incorporate a methodology based on the dynamic response of the floor in terms of velocity and acceleration, including a check for d_1 .

The review paper of Hu *et al.* seems to close a first period in the research field of timber floors vibration coinciding with the World Timber Conference in Engineering of 2004 and the release of the Eurocode 5. The next five years till the beginning of the new decade can be seen as a transition period in which no major breakthrough is achieved, however, it can be appreciated how researchers started to adopt recent technologies related to structural dynamic analysis as Operational modal analysis (OMA) or FEA. Hence, research on timber floor vibrations began to adopt methodologies and procedures of the aforementioned research on floor vibrations developed in the U.K. and at that time already consolidated in the design guides of Figure 2.19.

2.4 Vibration control

Vibration control is the research field focused on mitigating the dynamic response of structural systems subjected to dynamic excitation. This main objective might be accomplished in a wide variety of ways. Probably, the most basic one would be trying to get rid of the dynamic excitation, or if not possible, trying to reduce, modify or change the final dynamic excitation entering into the structure. This can be achieved by using an intermediate element between the structure and the excitation. This first strategy, not always applicable, is commonly known as 'isolation'. In the case of floor vibrations,

isolation is very convenient when the vibration receiver does not have to coexist with the source of vibrations (for example, in a building with a sports centre and a library, isolating the first one from the second would be advisable). If that coexistence needs to happen (for instance, in an office floor with seats and corridors where working people share the space with people walking and exciting the floor) isolation is not an option.

When isolation can not be considered, reducing the dynamic response of the concerning structural system, (modelled in its simplest way as an SDOF) is a matter related to the wise management of three fundamental variables: i) the mass, ii) the stiffness and, iii) the damping. However, an additional and more sophisticated fourth option can be added to these three: the control of the external force entering into the structure by using external devices that can introduce counteract forces to the structural motion to reduce it.

The control of the Stiffness and the mass, are classical approaches that civil engineers have used to deal with the undesired dynamic responses, especially in the field of vibration serviceability. Both of these approaches usually imply an increment of the structural mass on the change of better dynamic performance, thus, increasing the amount of material used and the EC footprint of the final structure. A brief description of them is included below:

- *Mass control*: This one usually implies increasing the structural mass to reduce the dynamic response of the structure. The main advantage of this approach is that it enables decreasing any type of dynamic response (harmonic, resonant, random, impulsive) almost proportionally. This strategy may get very sophisticated when the mass is punctually increased at particular structural locations, to tackle certain vibration modes. The main drawback of mass control is that it usually entails a structural oversizing to resist the additional self-weight added.
- *Stiffness control*: This approach is usually applied when the nature of the excitation source is harmonic, and thus, it causes a problematic resonant response on the structure. In these cases, the affected natural frequencies can be raised by increasing the stiffness of the structure. Stiffness can also be added locally at certain points. This strategy may be very competitive when the stiffness can be increased by severely modifying the structural layout, for example, by increasing the number of supports or connecting the structure to stiffer elements. When this is not possible, stiffer structural elements (beams, columns, diagonals...) need to be used and the efficiency of this approach drastically decreases. In these latter cases, the increase of the

stiffness usually encompasses a substantial increment of the mass, which makes even more difficult the task of raising the natural frequencies.

Stiffness control also encloses other types of more complex semi-active applications in which the stiffness of certain structural elements is wisely changed in real-time to minimize the motion of another structural point that is measured with a sensor, as the one developed by Nemir *et al.*[150]. A relevant application in this sense is the development of the so-called AVS (Active Variable Stiffness) system, based on inter-story diagonal elements that could be locked or unlocked using a hydraulic circuit in real-time to minimize the story drift [151]. The concepts of active and semi-active systems will be defined in detail later.

When designing building floors, the efficiency of these approaches for solving VLSL problems is reduced, as the rigid architectural constraints in terms of column matrix and floor slenderness usually strain the use of heavily oversized floors. Despite this fact, these strategies are the most widely extended in the engineering practice.

The main alternatives to these approaches are the increment of the damping and the use of counteract force strategies. Usually, both of them imply the use of elements to which civil engineers are not so habituated: 'dampers', 'actuators' or 'control devices'. In many engineering industries, such as automotive, aerospace, energetic off-shore or seismic, these are widely used not only as solutions to fix a wrong dynamic behaviour but also as key elements at the design stage of the product. In these industries, it is common to find structural systems in which dampers (of all kinds) strongly interact with the structural layout or even serve as capital structural elements.

The purpose of this section from now on, is to provide a brief classification of all these damping and counteract force technologies currently available, or that could have any potential application to address VLSL problems in floors. Both motion control approaches are defined below:

- *Damping control*: This implies increasing the structural damping in terms of energy dissipated inside the own structure. Commonly, the energy dissipation is based on frictional, viscous, Viscoelastic or hysteretic mechanisms. The additional dissipation is usually concentrated in certain structural locations where the motion between two structural elements might be exploited to include a damper device. These strategies result in being quite useful for reducing broad frequency band dynamic responses, however, their, design is usually challenging as their structural integration requires

strong iterative work to come up with the right solution.

- *Counteract-force control*: This strategy is generally based on devices which apply counteract forces to the structural motion in real time. This type of control can be classified into two main groups:
 - Inertial control: These are devices that can be locally placed where excessive motion needs to be controlled. They apply counteract forces to mitigate the dynamic response through the movement of an inertial mass in the opposite phase to the structural movement. Within the civil engineering context, this technology is commonly represented by tuned mass dampers (TMDs). The main particularity of this inertial controller is that, usually, they need to be tuned to the natural frequency to be tackled. Hence, their control capacity is considerably effective, but in a narrow frequency band.
 - Non-inertial control: These are active or semi-active devices, that in a variety of ways, enable the application of counteracting forces on the structure without using an inertial mass. They are usually represented by linear actuators or elements able to change their mechanical properties in real time at will. The mitigation capacity of some of these technologies, such as linear actuators, is outstanding, however, their implementation and design are challenging for civil engineers. Some other technologies are not mature enough nowadays to be implemented in civil engineering applications.

All of these types of motion control strategies can be classified according to their functioning principles into three categories: i) passive, ii) semi-active, and iii) active. The definition of these concepts is here provided:

- *Passive systems*: where the operation of all the structural and motion control elements is based on mechanical principles. Therefore, no control feedback system is implemented to modify the structure in real time.
- *Semi-active systems*: where the mechanical properties of some of the structural elements can be changed in real-time to minimize the dynamic response at a given point. This is done by measuring the dynamic response of the structure with sensors and implementing a feedback control system with a proper control law. Of course, specific elements able to change their mechanical properties need to be used. This type of strategy does not introduce external energy into the structural system, as no additional excitation is included.

- *Active systems*: where an external force is introduced in real-time into the structure to minimize its motion. Therefore, the use of actuator elements is basic for these implementations. A feedback control system with an adequate control law is again needed to accurately predict the required force at each particular moment. This strategy entails the introduction of external energy into the system to be controlled, so its energy consumption is considerably higher than the previous ones.

2.4.1 Damping control

This subsection digs deeper into the different damping control techniques currently available in the literature to be of applicability for reducing floor vibration.

Passive damping control

- *Material selection*: Some materials such as wood (3-4% of critical) or concrete (2%) can provide higher intrinsic damping values than others like steel (0.45%) or aluminium (0.6%). This is mainly because of the higher amount of non-linearities involved in the material mechanical behaviour, mainly related to frictional mechanisms developed into their microstructure.
- *Structural joints selection*: some types of structural joints as bolted joints tend to dissipate much more energy in terms of friction than others, such as welded joints.
- *Non-structural elements*: Partitions and non-structural elements are always an important source of structural damping in building floors.
- *Viscous dampers*: These are probably the most widely-known type of dampers. The dissipation mechanism is the development of viscous forces inside an embolus when the piston inside it tries to move. Their implementation has to be performed between two points with a prominent relative displacement between them. They are mostly used in seismic engineering and for damping cable vibrations in cable-stayed bridges. When designing them, the viscosity of the damper has to be well-adapted to the magnitude of the motion to be developed between the linked points. Their use in the context of vibration serviceability is limited [152] [153].
- *Viscoelastic Dampers*: VE materials have been mainly used in aircraft and mechanical engineering to mitigate undesired vibrations. Their use was then extended to civil engineering in the form of the well-known VE dampers. These

are devices located at certain structural locations where the relative movement between two structural points is significant. In those spots, the VE material of the damper can deform greatly to shear and hence, dissipate additional energy which is transformed into heat. These dampers have been widely used for cancelling seismic and wind-induced vibrations, as was done on the floors of the twin towers of the World Trade Center.

Additionally, VE materials can be applied on thin layers between two elastic members that vibrate in bending. When both elements vibrate the thin VE layer deforms to shear again, dissipating additional energy. This configuration is commonly known as Constrained Layer Damping, or CLD and it is the one studied in this thesis. The first commercial application of this system was developed by Willford *et al.*[8] to be applied to composite floors between the steel profiles and the concrete slab.

The design of VE damping control is difficult, as the shear stiffness of the VE element needs to be properly designed if optimum energy dissipation is wanted. Additionally, the mechanical properties of VE material vary with the temperature and the frequency of loading.

- *Frictional dampers*: Their layout is usually similar to one of VE dampers, but the energy dissipation is achieved through the friction between two elements. Their application in the field of VSLC is limited [154].
- *Shape Memory Alloy materials*: These are materials with an inherent superelastic nature that enables them to dissipate energy on a hysteresis. These materials are embedded inside the section of some structural elements and have been specially used for seismic applications.
- *Piezoelectric materials*: These are materials with an inherent ability to convert a difference in their stress level into a difference in electrical potential. Some contributions have studied their application as bonded layers on the surface of small structures. The material is then connected to an electrical circuit in which the energy transformed by the piezoelectric material is dissipated.

Semi-active damping control

- *Smart dampers*: This name encloses many types of damper technologies that enable changing their mechanical properties (viscosity, stiffness...) at will. Some of them are: i) magneto-rheological dampers, ii) electro-rheological, or iii) variable orifice dampers. All of these can adapt their properties in real-time to improve their

damping capacity. Real-time means that the damper can change its properties several times within a cycle of the higher frequency to be controlled. The resulting impact on the structure may be damping or stiffness changes and depends on the assumed control law. Since the response of these dampers is usually highly nonlinear, on/off strategies are commonly used due to their simplicity and effectiveness. This technology is most commonly applied in seismic engineering. Its direct use for damping floor vibrations has not been researched. Instead, they have been widely employed for the development of semi-active inertial controllers, as will be explained later.

- *Variable frictional joints*: There exist several examples, specially developed within spacecraft engineering, of rotatory joints on-off controlled through increasing or decreasing their friction in real-time (stiffening or loosening them), to maximize the amount of energy dissipated.

Active damping control

- *Active Constrained Layer damping*: This is the only application developed that can be classified into this group. Its configuration is similar to the one of a VE CLD, but the constraining layer used is a piezoelectric actuator that can be contracted or extended as desired to maximize the shear deformation of the VE layer. Controlling in real time the piezoelectric constraining layer, the damping to be obtained can be maximized.

2.4.2 Counteract-force control

This subsection explored the field of counteract-force control technologies currently available to reduce floor vibrations.

Inertial Counteract-force control

These devices, also known as tuned vibration absorbers, tuned mass dampers or TMDs, cancel the structural vibration by applying counteracted forces in the opposite phase to the structure displacement. That is, the structure applies forces to the inertial controller and the controller reacts by applying forces to the structure. They usually consist of a mass attached to the controlled structure through springs and dampers. In addition,

more sophisticated semi-active TMDs (STMDs) and active mass dampers (AMDs) can be integrated by making use of smart dampers and/or actuators for performing vibration control.

- *Passive Inertial counteract-force control - TMDs*: These were first conceived by Den Hartog in 1956. They consist of a mass-spring-damper system attached to the structure and tuned to a problematic vibration mode by correctly sizing their mass, spring and damper elements. The value of the inertial mass is chosen to achieve a vibration reduction objective once the structure is under resonance. The relationship between the inertial mass and the spring elements sets the tuned frequency (which is the key point to achieve the final performance). The damper makes the device's design robust when considering a certain degree of uncertainty related to the natural frequency of the targeted mode, as this one might change over time once the TMD has been installed. De-tuning of the TMD deteriorates significantly its vibration control performance. Hence, the trade-off between robustness and damping capacity must always be carefully considered when designing TMDs. This is, by far, the most widely used technology for damping serviceability vibrations in footbridges as floors. Their application on floors is usually done with the so-called Cantilever TMDs, where mass hangs from a cantilever tip, thus reducing the vertical free space needed for their installation.

In floor structures, it is common to use a single TMD for multi-modal control. In these cases, the TMD should then be tuned between the tackled modes; nevertheless, the design is not obvious and its control performance might decrease sharply. If this happens, a multi-TMD control might be used. This option is mainly used to cancel several spatially coupled modes with similar natural frequencies, as usually happens in modern floor layouts. Nonetheless, great care should be taken to the design and placement of the controllers since the interaction between the forces coming from the different TMDs might not be helpful in terms of vibration reduction. That is, the control action at one point can be interpreted at other points as a perturbation in such a way that the control action of one TMD can be destroyed by the others. In such a case, the control system would not be beneficial at all.

- *Semi-active inertial counteract-force control - STMDs*: These are mainly used for multi-modal control and to solve de-tuning problems that affect TMDs. They consist of an attached spring-mass-damper system capable of adapting its stiffness or damping in real-time to the relative motion between the controller and the structure. That is the reason that explains the use of smart dampers for developing SMTDs.

These changes are carried out to keep the controller tuned to the harmonic response of the structure. Lately, for performing these changes, on-off phase control laws have become popular due to their simplicity and efficacy. These systems can cope with harmonic vibrations coming from different vibration modes, which makes them effective under relevant structural uncertainties. Moreover, these devices enable the designer to deal with multimode dynamic responses or vibration modes with time-varying modal properties that could not be easily addressed by making use of several TMDs. However, they are not effective when the structure is not responding harmonically; that is, if several modes well separated are relevant in the response, the inertial mass does not vibrate harmonically, and their efficacy is again limited.

- *Active inertial counteract-force control - ATMDs.* They consist of an actuator inserted into a mass-spring-damper system, which actively moves the mass in real-time to produce counteract forces that cancel vibration, being generally much more effective than their passive and semi-active versions. Unlike them, ATMDs enable the control of broad frequency-band vibrations, and what is indeed more important, they can cope with non-pure harmonic responses since the movement of the inertial mass is controlled by the actuator. Its main disadvantages are the maintenance costs required for its functioning, and that a continuous power supply is needed. These devices were first conceived to be used in seismic applications. Their application to the field of floor vibrations was studied for the first time by Hannagan. Since then, important research has been performed by Diaz. The selection of the type of actuator is crucial as this one needs to be capable of reproducing the dynamics required for the control. In floors, the most typical actuators used are electrodynamic ones. Recently, there has been a technical breakthrough in the use of this technology, as the first company which commercialise this type of solution was created in 2022, and it is called CALMfloor.

Non-inertial counteract-force control

These systems consist of actuators that directly introduce counteract forces into the structure at a very specific location. They have been mainly applied in seismic engineering. Their use is not popular for reducing floor vibrations. Additional applications of this type have been developed with actuators that apply tension forces to cable elements in real-time.

2.5 VE materials

2.5.1 VE polymers for vibration damping

To understand the VE mechanical behaviour of typical materials used for damping control applications, it is capital to comprehend their internal microstructure. Polymers are materials made of macro-molecules forming long chains built from the concatenation of smaller sub-units called '*monomers*'. The internal microstructure of a VE polymer is usually composed of three elements as can be seen in Figure 2.21: i) long molecule chains, ii) covalent links between these chains, and iii) entanglements between chains. Those polymers with an orderly distribution of chains are called crystalline, while those with a messy network of chains are known as amorphous. These latter are the ones mainly used for vibration-damping applications.

Additionally, depending on the level of cross-linking (the amount of covalent links between chains), polymers can be divided into three groups: i) thermoplastics: with a low level of cross-linking, ii) elastomers: that form networks with a low level of cross-linking, iii) Thermosets: with dense meshes of cross-linked chains quite entangled. Usually, but not always, polymers used for VE vibration damping, such as rubber materials, belong to the thermosets or thermosets-elastomers classes.

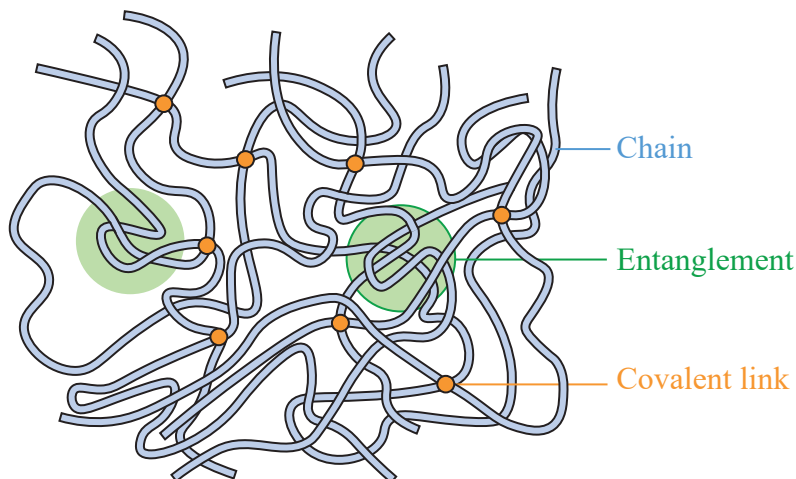


Figure 2.21: Elements of the microstructure of a VE polymer.

On the one hand, the presence of the covalent links between chains creates a sort of cross-linked network that confers elastic mechanical properties to the material. Hence, when this one is loaded, the network compresses or stretches instantly. On the other hand,

the presence of entanglements provides a kind of viscous behaviour. These entanglements are like knots between the chains where 'frictional' forces (or secondary valence forces) difficult the relative movement between each other. However, over time, some of these entanglements unravel or are overcome, and the network continues deforming.

This conceptualization enables explaining the two main mechanical behaviours that characterize VE materials: relaxation and creep:

- Relaxation: is a characteristic of some materials by which the stress level of a material decreases with time when a certain strain level is imposed. This is explained by the progressive unravelling of entanglements which implies the reduction of internal forces opposing to the same strain level. The time it takes for a polymer to fully relax its stress level is usually called relaxation time.
- Creep: is the phenomenon whereby some materials continue to deform over time when a constant stress level has been imposed on them. In this case, the same analogy is applicable. As the secondary valence forces are overcome by the chains' flow caused by the externally imposed load, this one continues to deform.

The mechanical behaviour of a VE material changes with the temperature. At lower temperatures, the energy of the molecules is really small and their mobility is negligible. In this situation, the secondary valence forces between them are high enough to confer the material a glassy structure characterized by an almost elastic mechanical behaviour. When the temperature increases, the energy received by the chains is higher so they have more molecular mobility. This causes the breakage of some secondary valence forces and the loosening of certain entanglements, which implies the end of the glassy behaviour. This happens at a temperature called the glass transition temperature, T_g . At this point, the VE material exhibits a rubbery VE behaviour and its dissipation capacity is maximized. The VE materials used for vibration damping work in this region. Finally, at higher temperatures, the material ends up being soft and deformable as the chains simply slide without any restriction, due to secondary forces have been completely broken and entanglements unravelled. From this, it can be deduced that the higher the temperature, the lower the relaxation timer of a polymer.

To conclude, it is important to remark that for VE polymers, there exists a correlation between their mechanical behaviour at different temperatures and at different harmonic frequencies of motion. When these frequencies are high and the period of vibration is considerably higher than the relaxation time of the polymer, its molecules do not have

enough time to transmit the energy to each other. In this case, the material hardly deforms and the dissipation of energy is negligible, so it behaves as in its glassy state. On the counterpart, when the frequency is low and the period is far below the relaxation time, the material can fully relax between cycles, achieving higher strain levels but again not dissipating much energy. Finally, when the period of motion is similar to the relaxation time, its behaviour is similar to the rubbery transition one, in which the dissipation capacity is maximized.

2.5.2 Time-domain mechanical behaviour

To accurately model the mechanical behaviour of VE materials in the time domain, attention has to be paid to modelling the phenomena of relaxation and creep which have been depicted in Figure 2.22

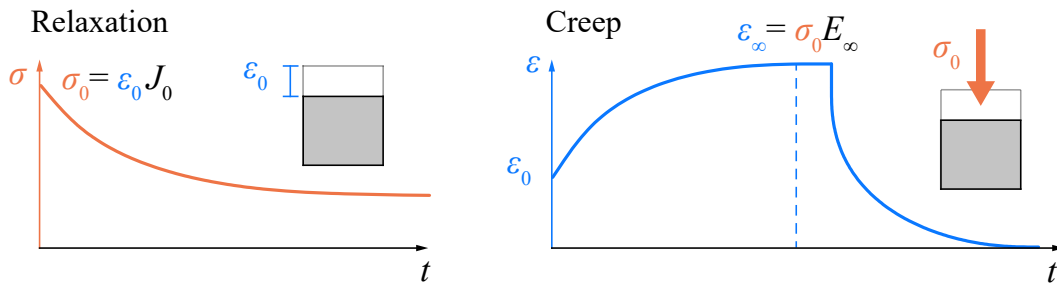


Figure 2.22: Creep and relaxation phenomena that characterize the time-domain mechanical behaviour of VE polymers.

Therefore, focusing on a uni-dimensional model of VE material, when this one is initially subjected to an imposed strain level ϵ_0 , the level of stress developed with time $\sigma(t)$ is modelled as:

$$\sigma(t) = \epsilon_0 (\phi(t) + E_\infty) \quad (2.25)$$

where $\phi(t)$ is the time-dependent part of the modulus between stress and strain and it is commonly known as the 'relaxation function'. E_∞ is the elastic modulus at $t = \infty$. This means that $\phi(t)$ is a function that decreases with time and it meets that $\phi(\infty) = 0$. In linear VE models the values of $\phi(t)$ and E_∞ are independent of ϵ_0 .

Moving on through creep, when imposing a constant stress level σ_0 , the strain obtained

on the VE with time is related to the first one as follows:

$$\epsilon(t) = \sigma_0 (J_0 + \Psi(t)) \quad (2.26)$$

where $\Psi(t)$ is the 'creep function' and J_0 is the instantaneous compliance at $t = 0$. Thus, $\Psi(t)$ needs to meet $\Psi(0) = 0$.

Finally, it is important to understand that VE materials usually meet the commonly known Boltzmann Superposition principle illustrated in Figure 2.23.

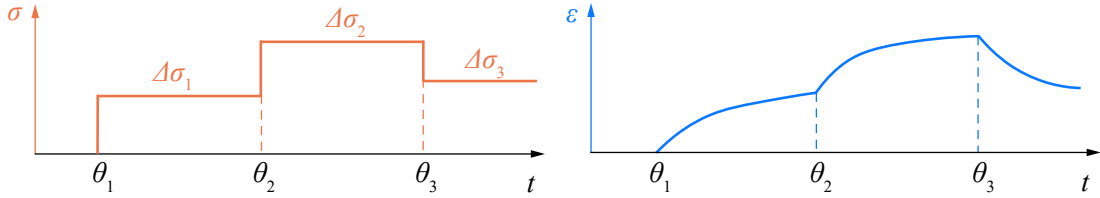


Figure 2.23: Boltzman superposition principle

This one states that the strain produced in time by any set of applied stresses is the sum of the individual strains produced by the individual stresses. Applying this principle implies that the current situation of the material depends on its history of stresses and strains. By applying the superposition principle, the following equations can be obtained in their discrete and continuous forms, for the creep and the relaxation phenomena:

$$\epsilon(t) = \sum_{\theta_i=-\infty}^{\theta_i=t} (J_0 + \Psi(t - \theta_i)) \Delta(\sigma(\theta_i)) = \int_{\theta_i=-\infty}^{\theta_i=t} (J_0 + \Psi(t - \theta)) \frac{d\sigma(\theta)}{d\theta} dt \quad (2.27)$$

$$\sigma(t) = \sum_{\theta_i=-\infty}^{\theta_i=t} (E_\infty + \phi(t - \theta_i)) \Delta(\epsilon(\theta_i)) = \int_{\theta_i=-\infty}^{\theta_i=t} (E_\infty + \phi(t - \theta)) \frac{d\epsilon(\theta)}{d\theta} dt \quad (2.28)$$

where θ_i represents in the discrete domain, the times at which a variation of stress $\Delta(\sigma_i)$ or strain $\Delta(\epsilon_i)$ occur. θ is the same parameter but in the continuous domain. Solving these integrals from $-\infty$ to t , two-time domain relationships between $\sigma(t)$ and $\epsilon(t)$ can be obtained based on the relaxation and creep functions, which in fact, are related between each other.

2.5.3 Frequency-domain mechanical behaviour

The frequency-domain mechanical behaviour of VE materials is intrinsically related to its time-domain one. In fact, by applying the Laplace transform and convolution theorem to Equation 2.29, and assuming the Laplace parameter $s = i\omega$ a relationship in the frequency domain can be obtained between stress and strain:

$$\sigma(\omega) = E^*(\omega)\epsilon(\omega) \quad (2.29)$$

where $E^*(\omega)$ is a complex elastic modulus that varies with the frequency of motion.

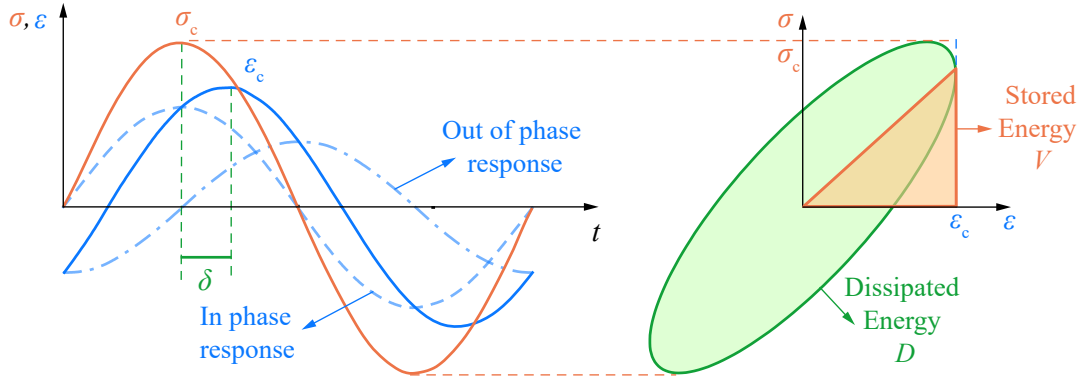


Figure 2.24: Harmonic loading on a VE polymer.

The meaning of this Equation is explained below on the basis of a cyclic loading test. If a piece of VE material is harmonically loaded with a stress $\sigma(t) = \sigma_h \sin(\omega t)$, the strain response of the material will be also harmonic $\epsilon(t) = \epsilon_h \sin(\omega t - \delta)$ but phase-lagged an angle δ , as can be appreciated in Figure 2.24. This phase lag results in the creation of a strain-stress hysteresis loop between $\sigma(t)$ and $\epsilon(t)$, and the area enclosed by the loop is the amount of energy dissipated per unit of volume within the VE material. Thinking of complex numbers in polar coordinates, the modulus of the complex young modulus can be then computed as $|E^*| = \sigma_h/\epsilon_h$, and its phase will be δ . Thus the complex modulus of the VE material at that particular excitation frequency ω can be expressed using the imaginary unit $i = \sqrt{-1}$ as follows:

$$E^* = |E^*| (\cos(\delta) + i\sin(\delta)) \quad (2.30)$$

From here, two elastic moduli can be obtained. First, the so-called storage modulus,

$E' = |E^*| \cos(\delta)$. This one is the elastic part of the complex modulus, responsible for creating the contribution of the final response which is in phase with the excitation. Second, the dissipative modulus $E'' = |E^*| \sin(\delta)$ which corresponds to the viscous contribution of the VE material and generates the contribution of the final response out of phase with the excitation. The complex modulus can be therefore expressed as follows:

$$E^* = E' + iE'' \quad (2.31)$$

The quotient between the dissipative modulus and the storage one (the tangent of the phase lag *delta*) is commonly used as a measure of the dissipative capacity of a VE polymer. This parameter is called the 'loss factor' η of the VE material. The higher this parameter, the higher the dissipation capacity of the material. η is calculated as indicated below:

$$\eta = \tan(\delta) = \frac{E''}{E'} \quad (2.32)$$

After all, the most common way of expressing the complex modulus of a VE material is the following one, using the storage modulus and its loss factor:

$$E^* = E' (1 + i\eta) \quad (2.33)$$

Additionally, it is important to understand, that in a VE material, the loss factor η , can also be computed as a quotient of two energies. The dissipated energy on one vibration cycle D (the green area inside the ellipse of Figure 2.24), and the maximum stored energy by the VE material on one vibration cycle V . The two of them can be computed as described hereon:

$$V = \frac{\sigma_c^2}{2E'} = \frac{E' \epsilon_c^2}{2} \quad (2.34)$$

$$D = \frac{2\pi\eta\sigma_c^2}{2E'} = \pi E'' \epsilon_c^2 \quad (2.35)$$

As stated before, the loss factor η and the modulus of the complex modulus $|E^*|$ depend on the frequency of motion and the temperature of the polymer. This dependency in the frequency domain has the appearance shown in Figure 2.25

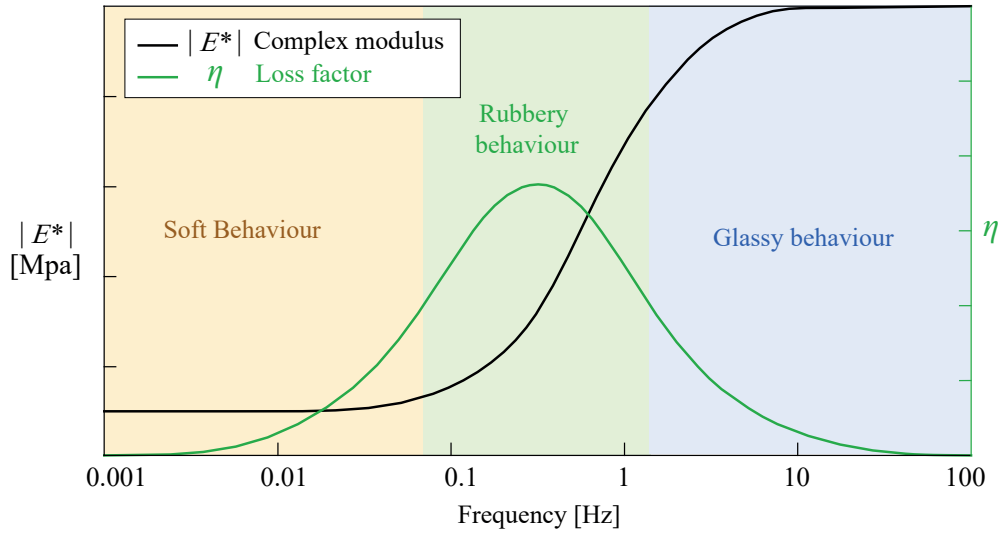


Figure 2.25: Typical appearance of the dynamic mechanical properties of a VE polymer as a function of the frequency.

2.5.4 Temperature-frequency analogy in VE polymers

As commented before, when studying the mechanical behaviour of VE polymers, an analogy can be made between the mechanical behaviour of the polymer at different temperatures and different harmonic frequencies. This means, for example, that a given polymer may have the same complex modulus E^* when subjected to a high-frequency motion at a high temperature, and when subjected to a low-frequency motion at a lower temperature. This correspondence between temperature and frequency has proven to be satisfactory, and indeed it is commonly used to experimentally obtain the dynamic mechanical properties of polymers for different temperatures and frequencies. The main idea under this is to test the polymer on a restricted frequency band at different temperatures, and then, shift the results obtained in the frequency domain to concatenate them as has been illustrated in Figure 2.26 for the modulus of the complex modulus $|E^*|$. For that, shift factors s are used for each curve.

This analogy is especially convenient for plotting the mechanical properties of VE polymers as a function of temperature and frequency in a specific type of abacus called nomograms. An example of this type of chart is included and used in Chapter 5 of this thesis 5.3.

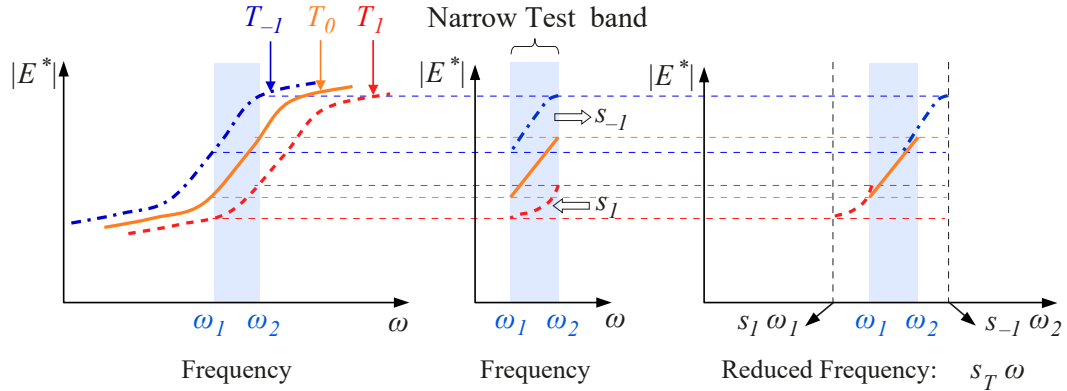


Figure 2.26: Temperature frequency analogy in VE polymers [7]

2.6 VE treatments for damping flexural vibrations

VE materials have been successfully applied as thin layers or ‘tapes’ glued to a bending vibrant surface on a wide variety of mechanical systems. In these cases, they are intended to dissipate energy through normal or shear stress loading cycles. Hence, two types of treatments may be differentiated:

- **Unconstrained Layer Damping (ULD):** consists in attaching a VE layer to an eccentric surface of the structural element to be treated. ULD enables additional energy dissipation through extensional hysteresis of the VE tape. The additional damping achieved with this technique is usually small (Figure 2.27.(a)).
- **Constrained Layer Damping (CLD):** consists in constraining a layer of VE material between two bending elements. In this case the layer must be located as close as possible to the sectional centroid of the element to be treated, as the energy dissipation is achieved through a shear hysteresis. This treatment usually provides greater damping improvements than the ULD. It is important to note that in CLD treatments there exists an optimum geometry (thickness and width) of the VE layer that provides a maximum damping increase (Figure 2.27).

Both damping treatments are depicted in Figure 2.27 which also includes the rules of thumb for efficiently designing them. This thesis is about the integration of integration of CLD treatments into composite floor systems.

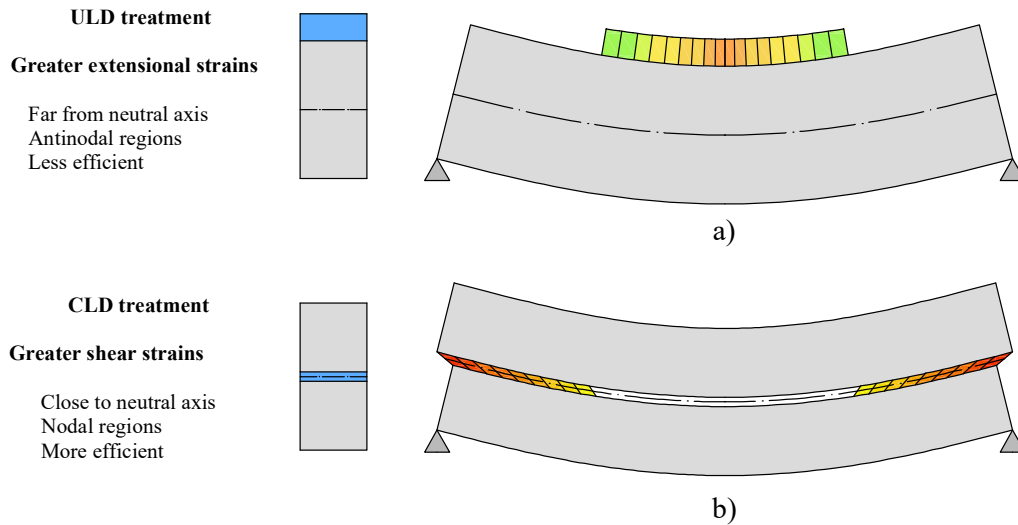


Figure 2.27: Conceptual explanation of CLD and ULD treatments

2.6.1 CLD treatments to control floor vibrations

CLD treatments have been extensively used in aerospace and mechanical structures to reduce broad-band frequency vibrations [7], however, their use in civil engineering structures is still limited [155], [156]. The initial applications of CLD treatments to composite floors to increase their damping ratio were developed as retrofitting solutions (Figures 2.28a, [157], and 2.28b, [158]) achieving damping ratio increases of 3.5% to 5%. Later, investigations were conducted by Ebrahimpour and Ahmadi, Goodchild, Fuller, *et al.* obtaining similar results, but with different CLD configurations (Figures 2.28c, [159] and 2.28d, [160]). In 2006 ARUP in collaboration with Richard Lee Steel Decking proposed the first commercial CLD solution to be integrated into a composite floor since the construction stage. This was called “Resotec” (Figure 2.28e), and its applicability was described by Willford, Young, and Algaard [8]. The system was conceived to be applied to the secondary beams of a composite floor as depicted in Figure 1.5. The major advance of this proposal over the previous ones was to locate the VE layer between the rib deck concrete slab and the upper flange of the steel beam (as close as possible to the composite section centroid) where the shear strain is maximum. Besides, they recommended including the VE layer only for a percentage of the beam’s length near the supports, where the longitudinal shear strain is higher, whereas the central part of the composite beam remains connected to shear using studs. Therefore, there is a trade-off between the achieved inherent damping and the loss of stiffness due to the partial shear connection in length. The CLD treatment itself used by ARUP has an overall thickness

of 3mm and is composed of two thin steel sheets of 1mm that constrain a slim VE layer of 1mm. The present thesis is focused on studying CLD treatments similar to the one proposed by Willford, Young, and Algaard.

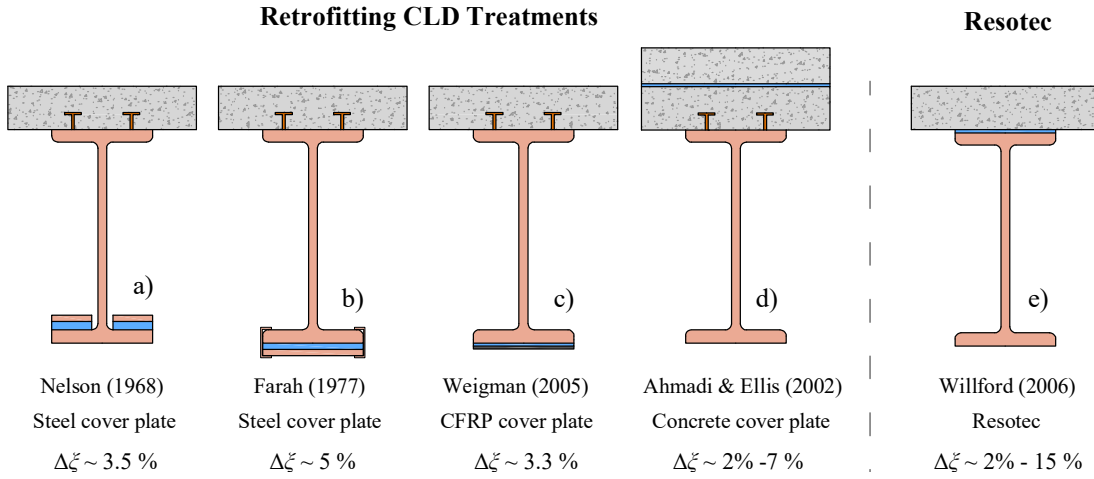


Figure 2.28: Evolution in the application of CLD treatments to composite floors.

