1. Introduction

As strength of materials and construction techniques improve, stiffness and mass rather than strength of structural elements have the potential to nowadays dictate layout of floors and sizes of their elements. This is clearly manifested through vibration serviceability which is becoming a governing limit state and design criterion for floors in buildings. Long and sufficiently strong floors are becoming lively and increasingly failing vibration serviceability under human-induced dynamic loading, such as normal walking.

However, awareness of this fact is still not widespread in the civil structural engineering community worldwide. Research of the problem and pertinent design guidelines are often lagging what is done in practice under increasing commercial pressures, and there is very little training in the subject in universities and through continuous professional development. As a consequence, there has been considerably more floor vibration serviceability problems reported in the last decade than in the previous century [1].

Having this in mind, the paper focuses on one of the most important unresolved problems in floor vibration serviceability nowadays: the effects of vertical partitions and cladding on floor vibration behaviour. Although such effects have been observed since mid-1960s [2], there is still a lack of fundamental understanding and they remain difficult to model during design despite potentially huge benefits to floor vibration performance. Instead, when faced with potentially poor floor vibration performance, the typical design approach is to add mass and/or stiffness, and/or increase damping to control floor vibrations. Adding columns, reducing spans, increasing beam/column sizes and slab thickness as well as employing materials which provide more damping (eg. using heavy concrete instead of light steel) are well known design techniques. However, these techniques are expensive, wasteful and environmentally unsustainable methods of passively controlling floor vibrations.

2. Historical Overview

Contributions to structural frames' stiffness and strength by non-structural internal walls and external cladding have long been recognised in the field of earthquake and wind engineering. There are numerous papers describing the effects of in-fill walls, brick-walls and other partitions as well as cladding on the sway/torsional behaviour of the whole building dynamically excited by seismic or wind forces. However, it should be stressed here that there is significant difference between the influence of non-structural partitions/cladding on serviceability-related vertical vibration of floors and on earthquake- or wind-excited horizontal vibration of the whole building. The key difference is several orders of magnitude larger dynamic forces and structural responses (displacements, accelerations, strains, stresses, etc.) caused by earthquake and wind loading, compared to, say, forces due to a single person walking and exciting floor vertical vibrations relevant to its serviceability. In the case of even perceptible but tolerable vertical floor vibrations, excitation forces are often so small that the corresponding displacements are of the order of microns. For example, a typical office floor vibrating sinusoidally in a single mode having a natural frequency of 6Hz and an upper limit response factor of R=4 has a peak acceleration of $0.028m/s^2$, as per Equation 1.

$$a_{peak} = 0.005 * 4/0.707 = 0.028 m/s^2 \qquad \text{(Equation 1)}$$

The corresponding peak displacement is only 20 microns, as given by the following equation:

$$a_{peak} = 0.028/(2 * \pi * 6)^2 = 20 * 10^{-6} m \qquad \text{(Equation 2)}$$

Under such a small level of displacement there is conisderable likelihood that friction forces are not overcome in various types of connections, including connections between non-structural and structural elements in a building frame. This is the key rationale as to why non-structural elements could provide additional stiffness to the structural elements they are touching and why they are so difficult to model.

Therefore, considering the focus of this paper, this historical overview only covers literature pertinent to partitions/cladding and their effects on the vertical floor vibration serviceability, typically under human-induced (by walking) dynamic loading. Petrovic and Pavic [1] provided a comprehensive review of the relevant literature published before 2011 which is directly pertinent to the problem in hand. To avoid repetition, only the highlights from this review, which was co-authored by one of the authors of this paper, will be mentioned here.

Lenzen [2] was the first to point out the potentially crucial role that non-structural partition walls have on improving vibration performance of floors. He thought that this was through providing additional damping to the bare floor structure. This conclusion was based on an investigation of 46 real steel joist-concrete floors. Interestingly, only three of these floors were open plan with no partitions and only those three floors had developed problematic annoying vibrations under walking-induced dynamic loading.

In 1975, Murray [3] suggested that floors with partitions should be modelled to have modal damping ratios of between 10% and 20%. As Murray himself noted, these values should be used with caution until badly needed research in this field is carried out. Forty years later, that research is still very much needed and it is now well established that the suggested values of 10-20% are far too high. These values were based on the so called heel-drop test which has been proven to be unreliable due to the inappropriate single degree of freedom curve-fitting technique used on decaying floor response after the heel-drop. Heel-drop is a broadband impulsive excitation and the response to it contained multiple decaying modal contributions which together appeared to decay faster than the individual modal contributions thereby giving the impression of higher damping [4]. This example indicates the need for correct scientific methods when investigating the effects of non-structural partitions, preferably combining experiment and experimentally validated analytical modelling of the tested floor.

Apart from damping, in 1987 Pernica [5] conducted measurements of floor modal properties (natural frequencies, mode shapes and modal damping ratios) during three phases of building construction:

1) the bare floor phase,
2) the phase immediately after the main structural components and the exterior masonry walls were constructed, and
3) the final phase when floor system was completed and all architectural partitioning and cladding elements installed.

Pernica provided the first conclusive evidence of cladding/partitions adding not only mass (which is obvious) and damping, but also stiffness. For the particular types of cladding/partitions, this was demonstrated via considerable changes in the measured natural frequencies and mode shapes, but no attempt was made to model and quantify these effects in a manner usable in future design.

In 1988 Ohlsson [4] also recognised the significant positive impact that partitions could have on the vibration properties of floors. However, he used the term 'temporary partitions' arguing that as partitions are not permanent features in buildings, as they can potentially be changed by a tenant for different building usages, they should not be relied upon when ascertaining floor vibration performance in design. In other words: if a floor is having satisfactory vibration performance when bare, adding partitions can only help and neglecting their effects is a safe assumption in design. Historically speaking, this logic is probably the main reason why floor partitions were not more actively researched and their effects codified in design guidelines when determining floor vibration performance. This school of thought has been embeded firmly in industry ever since. But, also, this school of thought from a quarter of centuty or so ago, has a number of flaws. Firstly, there are many types of buildings where installed partitions will remain for a good part or the whole of their intended design life, such as hospitals and schools. Secondly, when one partition layout changes it often gets replaced by another partition layout, rather than an open plan no-partition layout. Thirdly, the potential beneficial effects of partitions on floor's vibration performance could be so significanat that neglecting them (and, instead, adding more structural material/elements to ascertain vibration performance) is overly expensive, potentially wasteful and, in this day and age of required sustainability of the built environment, simply - unsustainable. This is particularly the case for multi-storey buildings when even small material savings per floor become substantial when multiplied by a large number of floors. Finally, knowledge about the effects of partitions and their modelling can empower and inform designers and their clients to perform risk-benefit analysis of including (or excluding) the partitions in design calculations, while considering the intended use and required flexibility of the structure in question.

After 2011, Middleton and Pavic [6] in 2013 published a paper in which, similar to Pernica [5], they described results of modal testing on a full-scale composite steel-concrete floor structure in its bare (featuring only slabs and columns) and fully fitted out (featuring all cladding and partitions) states. These state-of-the-art multiple-input multiple-output (MIMO) modal testing exercises of the two floor configurations were nominally identical; featuring identical test grids, hardware and data acquisition parameters. The high quality test results demonstrated without any doubt that the stiffening effects of the partitions offset all additional mass of the fully fitted-out floor configuration and increased the natural frequencies of the first two modes of vibration by 10-15%. Interestingly, vertical static stiffness at the point where one of the the shakers was placed on the fitted out floor increased significantly by more than four times compared with the bare floor. However, no modelling aimed at simulating this significant feature was reported in the paper.

Indeed, papers describing both modal testing of real floors during one or more of their construction phases, such as those published by Pernica [5] and Middleton and Pavic [6], and FE modelling aimed at simulating the measured features are extremely useful to learn about the effects of partitions, but are also very rare. To the authors' best knowledge there have been only two papers of this kind. Firstly, Miskovic et al. [7] performed modal testing of *only* the fully fitted configuration and subsequent finite element (FE) modelling of two nominally identical floors at different levels of the same building. They found substantial differences in the two sets of modal properties which were attributed to the effects of the different partitioning layouts on the two floors. By tuning the FE model to match the measured modal properties they found properties of vertical springs used to simulate the effects of the full-height plaster and glass partitions. More recently, in 2013, Brownjohn et al. [8] found properties of shell finite elements used to model non-structural brickwork and cladding of a high-rise building by tuning FE models of floor vertical vibration to match modal properties measured before (with no partitions/cladding) and after (with partitions/cladding) the retrofitting of the building.

Apart from the overall severe lack of experimental data on modal properties corresponding to various phases of construction of as-built real life buildings, another problem is a sheer number of different types of non-structural internal partitions and external cladding. Ideally, each of these require investigation and development of *taxonomy* similar to the one used in earthquake engineering for non-structural components in buildings [9].

To address the gaps in knowledge, this paper describes a combined experimental and numerical investigation of modal properties of two nominally identical floors in the same multi-storey building without and with non-structural partitions and cladding. For civil structural engineering relatively rare but powerful, MIMO modal testing has been employed as well as the FE modelling, manual model tuning and formal model updating based on the measured modal testing results. The aim of this dual approach was to assess the ability to predict as-built modal properties using best engineering judgement in the FE model development and to determine a viable strategy for FE modelling of the specific type of partitions used.

3. Test Structure – Charles Institute Building

The Charles Institute on the University College Dublin (UCD) campus in Ireland is a four storey reinforced concrete frame office building (Fig. 1).

Fig. 1: Completed Charles Institute Building.

Structurally, the frame consists of two-way spanning flat slabs, 0.3m thick, supported by 0.4m square columns with a maximum bay size of 7.5m × 6.6m. The lateral load resisting system is made up of a number of reinforced concrete stairwells, lift cores and service ducts with wall thicknesses of 0.2m.

3.1 Non-Structural Elements

The contribution of two types of non-structural elements was investigated in detail: internal full-height partitions and external cladding panels.

3.1.1 Non-Structural Partitions

Each floor level is divided into state-of-the-art laboratories and office accommodation using lightweight non-structural partitions, a vertical section of the partition is shown in Fig. 2.

Fig. 2: Detail of Non-Structural Partition.

The mass of these partitions is approximately 47kg/m (per unit length, as opposed to height). An assessment of its stiffness is made for correlation with subsequent numerical models in which their effects are considered.

These partitions consist of two layers of gypsum boards, 25mm total thickness, attached to both sides of lightweight metal studs at 400mm centres which house an insulation material. The vertical stiffness of these can thus be written per Equation (3).

$$K_{\mathrm{Partitions}} = K_{\mathrm{Supports}} + K_{\mathrm{Gypsum}} \qquad \text{Equation (3)}$$

Taking the Young's Modulus of the metal studs as 200,000N/mm^2 and 1,700N/mm^2 for the gypsum boards (based on Maail [10] and Plachy et al. [11]), with a partition height of 3.0m, and appropriate cross-sectional areas (183.75mm^2/m for the metal studs and 50,000mm^2/m for the four layers of gypsum board) from Fig. 2, a value of 40,583kN/m per metre of (horizontal) length of the partition is an estimate of its expected vertical stiffness.

3.1.2 Cladding Panels

Exterior wall cladding consists of large polished Chinese black basalt panels 40mm thick supported on galvanised steel rectangular sections fixed to the concrete slab above and below at 400mm centres, as detailed in Fig. 3.

Fig. 3. Typical Detail of Cladding panels and support system.

The polished basalt has an estimated mass per unit length of the cladding of 430kg/m.

Considering the presence of the 20mm movement joint between the stone elements, sealed with silicone (Fig. 3) between the basalt units in the vertical direction, it was assumed that additional vertical stiffness could be provided only by the steel frame sections. The height of these frames is 3.0m and, with vertical supports of $605mm^2$ cross sectional area at 400mm centres, the vertical stiffness is expected to be of the order of 101,000kN/m per every metre length of the cladding.

3.2 Floor Layout

The structural systems of the two floors are identical, only the layout of internal partitions varies from floor to floor. The layout of partitions for the two floors tested is shown in Fig. 4 and Fig. 5.

Fig. 4. Floor Level 1 internal partition layout.

Fig. 5. Floor Level 2 internal partition layout.

In both cases the solid black lines indicate the partition layout on the floor below. The only difference is that Floor Level 2 includes a staircase opening for a stair starting at Floor Level 1. However, this region is not the dominant area of the lowest vertical modes and the corresponding vibration response, so the two floors, being identical elsewhere, are considered *nominally* the same.

4. Operational Modal Analysis of Bare Frame

Operational Modal Analysis (OMA) was carried out on the bare frame structure of the building during construction so that an FE model could be calibrated to as-built bare frame modal parameters. Vertical ambient vibrations were measured using the test grid shown in Fig. 6.

Fig. 6 Bare frame tests - sensor layout.

The sensors used had a sensitivity of 700mV/g and a dynamic range of ± 3g. Amplifiers were used to scale the signals which were then digitally recorded. Ambient responses at each sensor location were recorded for 10 minutes at a sampling rate of 12kHz. Datasets of such length are considered adequate for a structure expected to have natural frequencies in the range of 5Hz to 35Hz [12].

The recorded data sets were detrended and windowed (cosine tapered) to avoid signal processing 'leakage'. Data was then filtered using a low-pass filter at 40Hz, converted into the frequency domain using Discrete Fourier Transform (DFT) and further post-processed using Frequency Domain Decomposition (FDD). The first step of the FDD process is to calculate the Power Spectral Density (PSD) matrix of the multiple channel vibration data using a number of DFTs [13]. This PSD matrix is then decomposed using Singular Value Decomposition (SVD) which represents the PSD matrix as the product of three separate matrices, U, the unitary matrix, S, the matrix of singular values and V, a transpose matrix. SVD is a mathematical technique of data reduction which can better expose the various relationships among data sets [14–18]. The values within the singular value matrix represent n single degree of freedom models which describe the system in the frequency domain, where n is the number of channels of vibration data. The respective modal amplitudes at each singular value are extracted from the U matrix and form an approximation of the system's mode shapes. Strictly speaking, this process requires white noise excitation and a lightly damped structure [19]. However, it has also been shown to be equally effective provided that the input spectrum is quite flat, typical of ambient excitation, and excitation harmonics are absent.

The resulting singular value plot of the vertical acceleration response of Floor Level 1 is plotted in Fig. 7 indicating that the first natural frequency corresponds to the small peak at 10.7Hz.

Fig. 7 Singular value plot extracted from the preliminary test of the bare frame structure.

There are also clusters of peaks in the ranges of 12-14Hz and 21-24Hz, as well as peaks between and above these ranges. Clustering of modes at similar frequencies is a feature of structures with the repetitive geometry, such as the floor system tested between supporting columns and shear walls.

5. Numerical Modelling of the Bare Frame

A 3D FE model of the building (Fig. 8) was developed from final construction drawings, using the ANSYS FE software [20].

Fig. 8 FE model of the Charles Institute Building.

Shell elements (known as Shell63 elements in ANSYS) were used to model the slabs and shear cores while beam elements (known as Beam4 in ANSYS) were used to model the columns. The Beam4 element in ANSYS has six degrees of freedom at each node and is capable of tension, compression, torsion and bending behaviour. The Shell63 element in ANSYS also has six degrees of freedom and is capable of both bending and membrane behaviour. Such elements are generally considered 'thin' plates meaning that their thickness is small relative to their width and length. Slab-column and slab-wall connections were assumed to be rigid. The boundary conditions assumed in the finite element modelling of a civil engineering structure have a significant impact on the behaviour of the model. For this FE model, the translational degrees of freedom were fully restrained at ground level for all shear walls and columns to represent a hinged restraint. The building's pad foundations were founded on Dublin Boulder Clay which is well known to be stiff with a bearing capacity of up to 150kN/m^2 ([21,22]). Therefore, the 3D FE model was supported by rotation-free pin supports. Springs representing the soil stiffness were not considered necessary.

The two way spanning floor slabs were assumed to be isotropic and were assigned a density of 2400kg/m^3. The concrete mix for the building was C30/37 per Eurocode 2 [23] and initially a Young's modulus of 37GPa was assigned based on this specification. Neville [24] concluded that the Poisson's ratio for concrete is dependent on the properties of the aggregate used but that it is typically in the range 0.17 to 0.20. Mehta and Monteiro [25] confirm this in recommending a similar range of 0.15 to 0.20 and Bamforth et al. [26] recommend 0.20. In this case natural frequencies were found not to be overly sensitive to the Poisson's ratio and the results reported are for a value of 0.18. The FE model was tuned manually, by varying only the Young's modulus value, until the first three Level 1 floor natural frequencies correlated to within 6% of measured values with Modal Assurance Criterion (MAC) [27] values greater than 0.8. In this way the Young's modulus value was updated from 37GPa to 48.5GPa. This is consistent with Mehta and Monteiro [25] who state that the dynamic modulus of elasticity for medium strength concrete is up to 30% higher than its static modulus.

The FE-predicted vertical floor modes were found to lie in the 10Hz to 35Hz range. A comparison of the natural frequencies and modes of vibration, based on MAC values, is given in Table 1 for Floor Level 1.

Table 1 Comparison of natural frequencies of floor Level 1 for the bare frame structure and updated FE model.

The FE model captured the first three frequencies and modes quite well. While direct correlation at higher frequencies is not as good, the presence of multiple modes in the same frequency range as measured had been replicated by the FE model which was encouraging. As previously mentioned, this would be considered typical of structures with repeated or slightly varying geometries. The first (experimental and FE) mode shapes for Floor Levels 1 & 2 are plotted in Fig. 9 and it is noted that the lowest FE modes for the two floor levels in the bare frame structure look very much the same.

Fig. 9: Experimental and FE mode shapes for Floor Levels 1 & 2:

(i) Floor Level 1 - OMA Mode 1 @ 10.7Hz;

(ii) Floor Level 1 - FE model Mode 1 @ 11.1Hz; and

(iii) Floor Level 2 - FE model Mode 1 @ 11.1Hz.

6. FRF-based Modal Testing - Floor Levels 1 and 2

Subsequent to the bare frame testing, FE modelling and model tuning, construction work was completed which provided an opportunity for the two floor levels to be tested again in their completed conditions. In contrast to the bare frame tests, all external cladding, internal partitions and furnishings were in-situ and the building was ready for occupancy. Multi-shaker forced vibration modal testing was based on Frequency Response Function (FRF) measurements on Floor Levels 1 and 2. It was carried out over two consecutive days while the building was unoccupied. A relatively dense grid of measurements was utilised, comprising 195 measurement points

(Fig. 10), to allow for a high resolution of mode shapes and to minimise spatial aliasing of the mode shapes extracted.

Fig.10: Detailed test grid for forced vibration analysis.

Only the vertical response of each floor level was measured using piezoelectric accelerometers. Excitation was provided by four electrodynamic shakers located at test points marked 1 to 4 (Fig. 10), so arranged to ensure an even distribution of excitation energy. The four shakers were driven by a 24-channel 24-bit digital spectrum analyser using uncorrelated random signals. FRFs corresponding to four shakers as references were determined for all measurement points and curve-fitted using a multiple-reference orthogonal polynomial algorithm, as implemented in the ME'scopeVES software [28]. These FRFs were then used to extract modal properties: natural frequencies, mode shapes and modal damping ratios.

The moduli of eight measured *point-mobility* FRFs [27] (whereby the excitation and response directions and points are the same) for both fully fitted-out floor levels and all four shaker locations, are plotted in Fig. 11.

Fig.11: Excitation Point FRFs for floors Levels 1 & 2.

The FRF peaks indicate that the natural frequencies ranged between 15.7-27.0Hz for Floor Level 1 and 15.0-25.0Hz for Floor Level 2. While these frequency ranges are similar, the floor point mobility FRFs are noticeably different. The first three modes of vibration, extracted from the measured datasets for Floor Levels 1 and 2 are shown side by side in Fig. 12.

Fig.12: Comparison of first three modes for each floor level extracted using FRF-based modal testing.

Fig. 11 and Fig. 12 demonstrate that the two fully fitted floors now have considerably different FRFs and modal characteristics. Compared to the initial experimental results on the bare frame structure (Fig. 9 and Table 1) it is evident that there is a stiffening of the floors from their bare conditions. For example, the first bending mode of Floor Level 1 has increased from 10.7Hz to 15.7Hz, an increase of over 30%.

7. Modelling of Non-Structural Partitions and Cladding

It is postulated here that the differing modal properties for Floor Levels 1 and 2 can be attributed to their associated internal partitions and external cladding systems, which vary between the two floors. To investigate this idea, the initially tuned bare frame FE model was modified to include internal partitions and external cladding systems modelled as linear elastic vertical springs and lumped mass elements. This revised FE model was then formally *updated* [29] to determine whether consistent properties for the partitions and external cladding systems could simulate the measured modal characteristics of the two fitted out floor levels.

The internal partitions and the cladding panels on the exterior of the building were modelled using vertical spring (Combin14) and lumped mass elements (Mass21). Combin14 element in ANSYS is a uniaxial tension-compression element with up to three degrees of freedom at each node. Bending or torsion/twisting stiffness of the partitions/cladding were neglected. To match the nodes of the existing beam and shell elements, the springs were placed at every 300mm along the partitioning support lines on each floor level. Combin14 elements were vertical and connected at points above and below each floor simulating full-height of both partitions and cladding. Mass21 element allowed inclusion of the mass related to both types of non-structural elements in the model. The configuration of springs used in the FE model, for Floor Levels 1 and 2, is shown in Fig. 13.

Fig. 13: (i) Floor Level 1 and (ii) Floor Level 2 of the FE model including springs representing partition and cladding elements.

The distribution of springs matches the geometry of partitions shown in Fig. 4 and Fig. 5, respectively, for the two floor levels. It should be noted that the whole multi-storey building was modelled in a manner similar to what is shown in Fig 8, but only Floor Levels 1 and 2 have been extracted from the whole building FE model and are shown in Fig 13 to make clear the presentation of the non-structural partitions/cladding as springs.

For the purpose of model updating, as previously mentioned, the mass modelling parameters (per metre length) of internal partitions (47kg/m) and cladding panels (430kg/m) were treated as constants. For all floor levels these masses were represented using lumped mass elements (Mass21) at the lower spring nodes on each floor level. In other words, partitions and cladding between Floor Level 1 and Level 2 had their respective lumped masses added to the slab at Floor Level 1. All partition springs were assumed to have the same stiffness, reflecting the use of identical partitions through the height of the building. Similarly, all cladding springs were also assigned the same stiffness which was different from the partitions' stiffness. During the model updating process only these two modelling parameters (i.e. two stiffnesses assigned to the vertical partition and cladding springs) were varied.

FE model updating was carried out using the Response Surface Method [29]. This method is based on generating 'response' surfaces for a given set of updating parameters - in this case the stiffness of the springs associated with the internal partition and exterior cladding elements. Initially, a matrix of natural frequencies, specifically the first six natural frequencies for each floor level, was generated by conducting 400 modal analyses. For these analyses the partition and cladding spring stiffnesses were varied between 0 and 60,000kN/m, in increments of 3,160kN/m per unit horizontal length of the partition/cladding. This range was broadly in line with a value of 5,429kN/m proposed by Smith and Vance [30] after conducting a number of seismic tests on internal partitions in laboratory conditions.

The natural frequencies calculated were then used to construct responses surfaces, using a dual variable 5th degree polynomial surface equation in Matlab, for each of the first six natural frequencies of both floor levels. The resulting response surfaces for Floor Level 1 are shown in Fig. 14.

Fig. 14: Dual variable 5th degree polynomial approximation of the natural frequency response surfaces for modes 1 to 6.

The response surfaces define the relationship between floor natural frequencies on one side, and varying partition and cladding spring stiffnesses on the other. The optimum solution for these spring stiffnesses is defined as those values that minimise the difference between experimentally measured frequencies (f_{EXPi}) and

FE model frequencies (f_{FEi}) for mode 'i'. The resulting spring stiffness values, which minimise the objective function, are listed in

Table 2.

Table 2 Stiffness values of partition and cladding springs as calculated using model optimisation.

Table 2 shows that the stiffness associated with the more robust external cladding elements is sensibly larger than that attributed to the internal partitions.

It should be mentioned here that the average floor vibration amplitude measured during the FRF measurements was 0.008m/s^2 which corresponds to a response factor R between approximately 1 and 2. This compares well with the target response factor for the floor operation (combination of laboratories and offices) of R=2 to 4. The FRFs measured were very stable after a small number of five averages and they also passed the *homogeneity* check of linear behaviour, meaning that within the frequency range of interest they were not very sensitive to the amplitudes of the excitation force. This is important as all civil engineering structures are in essence non-linear and have amplitude dependent behaviour over a wide range of amplitudes. It is therefore important to establish if, within the range of relevant floor vibration amplitudes, typically corresponding to the target response factors, the structure behaves linearly. It can be concluded that for the stated average vibration level of 0.008m/s^2 during the FRF-based modal testing both floors behave linearly and the established stiffnesses of springs correspond to that level.

The final step in the model updating process was to re-run the FE model with the two stiffness values identified. The resulting FE model natural frequencies are compared to those measured in-situ in Table 3.

Table 3 Comparison of experimental and updated numerical results**.**

The correlation between FE-calculated and measured natural frequencies is good, for both floors, with the maximum percentage difference being 10.9% while the average percentage difference, across 11 frequencies compared over the two floor levels, is only 3.8% with a standard deviation of 3.3%. Experimentally estimated and analytically calculated (using a multi-storey FE model as before) mode shapes for Floor Levels 1 and 2 are plotted side by side in Fig. 15 and Fig. 16**,** respectively. It can be seen that in addition to the satisfactory pairing of individual natural frequencies, the experimental and FE mode shapes compare well, too. Furthermore, the modelling strategy proposed is able to represent consistently the different modal characteristics of Floor Levels 1 and 2.

Fig.15: Floor Level 1 - FE calculated (left) and experimentally estimated (right) mode shapes.

Fig. 16: Floor Level 2 - FE calculated (left) and experimentally estimated (right) mode shapes.

8. Discussion of Results

The focus of this study has been the changes in the modal properties of two structurally identical concrete floors (one above the other in the same building) due to non-structural cladding and different layouts of non-structural full-height partitions. Whereas the two floors had the same modal properties when bare, the presence of cladding and partitions increased the fundamental frequency by 5Hz, from 10.7Hz to 15.7Hz i.e. by almost 30% for Floor Level 1. A similar increase was noted for Floor Level 2. However, although significant and similar, the two increases in the natural frequency were *different* for the two floors which can be attributed to the different internal partition layouts for each floor. This was proven by development of two updated FE models for both floors featuring cladding and different partition layouts modelled as linear elastic springs. During the updating, the exterior cladding system was found to require stiffer springs than the internal partitions as would be expected given the more robust nature of the cladding.

The internal partitions were found to require a spring stiffness of 11,000kN/m per spring to account for the change in modal properties relative to the bare frame system. In the FE model, springs were assumed at 0.3m intervals along the length of each partition and hence each 1m length of the internal partitions in this particular building could be represented by a vertical spring having stiffness of 36,360kN/m. For the exterior cladding, the resulting spring stiffness per metre length is 73,360kN/m.

The identified stiffness values shown in Table 4 are comparable to those calculated from the ideal section properties of the partitions and cladding presented in Section 3.1.

Table 4 Comparison of actual axial stiffness and axial stiffness calculated using model updating procedures.

The ideal section stiffness properties of partitions overestimate the FE-updated stiffness by 10%. The agreement with the FE-updated cladding stiffness is not as close - there is 27% overestimation when using ideal section properties. However, it is encouraging to see the FE-updated stiffness values lower than the theoretically calculated as this would be expected considering the imperfect connections pertinent to non-structural elements which are likely to manifest itself as lack of full engagement of the non-structural elements in the vertical direction i.e. a lower stiffness of an ideally connected spring. Nevertheless, the identified stiffness for both types of non-structural elements represent closely the physics of the non-structural items and substantiate the argument that the significant increase in the natural frequencies between the bare and fitted-out floors can in fact be attributed to the vertical stiffening effects of the cladding and internal full-height partitions.

Visual inspection of the experimentally estimated mode shapes can be used to interpret physically the results of the formal FE model updating. For the fitted-out system mode 1 of Floor Level 1 (Fig. 15), at 15.7Hz, is predominantly a bending mode occurring between gridlines 1-4 and A-I. Mode 1 of Floor Level 2 (Fig. 16), at 15.0Hz, is a similar bending mode but occurs over a broader area of the slab, between gridlines 1-4 and A-M. The difference can be attributed to the partition layouts (Fig. 4 and Fig. 5). Floor Level 1 has a partition along gridline I between gridlines 1-4, which constrains that mode, and also several of the higher Floor Level 1 modes. This would also explain the slightly higher frequency, 15.7Hz versus 15.0Hz, for Floor Level 1 compared to Floor Level 2. It is interesting to note that this partition at gridline I seems to contribute more to the Floor Level 1 (on which it sits) stiffness than to the Floor Level 2 (under which it is) stiffness. This is believed to be due to the presence of a further partition below Floor Level 1 at ninety degrees to gridline I rather than the fact that this particular partition is on Floor Level 1. Floor Level 2 does not benefit from partitions above and below in this region. Also, and more generally, the areas on both floors, between gridlines 1-9 (Fig. 11), most densely populated with partitions, are the least responsive in any of the mode shapes.

9. Conclusions

Modal testing of two floor levels of a reinforced concrete building has shown the considerable importance of non-structural elements, specifically internal partitions and exterior cladding systems, when analysing the vibration behaviour of building floors. Updated FE models have shown that the different vibration behaviour of two, otherwise identical, floor levels is due to the differing layouts of internal partitions used.

Conclusions, specific to the floor levels examined in this study, are as follows:

- in its bare frame state the modal properties of both floor levels are the same;
- once different partition layouts are added to each floor their modal properties differ;
- the impact of the non-structural components considered in this study, where the vertical response of the floor systems was studied, can be accounted for using simple linear spring elements in FE models;
- the partition and cladding spring stiffnesses were found to be consistent with the physics of both the internal partitions and the exterior cladding system; and
- the addition of non-structural elements result in a significant increase in floor fundamental natural frequencies, measured to be about 30% for the two floor structures investigated.

Based on these findings and subject to further research into other types of floors and partitions, there is a potential to utilise partitions as an effective and predictive means of passive control of floor vibrations considering the fact that they not only add mass and damping, but also considerable stiffness, as demonstrated in this paper.

Acknowledgements

The authors wish to express their gratitude to: (i) the Irish Research Council for Science, Engineering & Technology (IRCSET) grant no. RS2000810, (ii) the UK Engineering and Physical Sciences Research Council EPSRC grant ref. EP/G061130/2, (iii) Dr Christopher Middleton for his help with the technology used in this

work and in particular the field testing at UCD and (iv) the UCD Buildings Office for access to the Charles Institute.

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