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SEISMIC VULNERABILITY OF NON-STRUCTURAL COMPONENTS: FROM TRADITIONAL SOLUTIONS TO INNOVATIVE LOW-DAMAGE SYSTEMS

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Abstract: *Non-structural components are typically not designed for seismic loads, nevertheless their response can significantly affect the building functionality after earthquakes, even for low-intensity events. The poor performance of non-structural elements can result in substantial economic losses and business interruption. Consequently, damage of these components has severe impact in the post-earthquake building recovery in addition to the potential risk to life safety. Recent earthquakes have further highlighted these considerations and a substantial research effort has been dedicated to better understand the seismic behavior and damage states of non-structural elements and develop innovative solutions able to mitigate their risk of damage. In order to facilitate the quantification of damage levels as well as the proposal of practical and efficient low-damage solutions, a state-of-the-art overview on the seismic vulnerability and expected performance of alternative systems, either based on traditional construction practice and on innovative systems, is needed. In this paper, a collection of key characteristics of different non-structural components (facades, partitions, ceilings) in terms of seismic response, mechanisms and fragility curves, developed through laboratory testing, earthquake damage data and/or numerical studies available in literature, is presented. Finally, numerical investigations are performed in order to define and compare the fragility functions of traditional and low-damage solutions. Specifically, Incremental Dynamic Analyses are performed for a multi-storey case-study building with alternative precast concrete cladding configurations. Fragility curves are derived and a quantitative risk assessment is developed to highlight the efficiency and benefits related to the application of innovative low-damage technologies.*

Introduction

Non-structural elements are generally not considered part of the main load-bearing system of a building or industrial facility whilst are mainly designed for architectural performance, such as thermal, acoustic or fire. Although they are not included in the seismic design process, non-structural systems may be subjected to large seismic demands, depending on their own characteristics.

Non-structural elements include all those components not part of the primary structure, namely: 1) architectural components, i.e. façades, partitions, ceilings; 2) building utility services, i.e. mechanical and electrical building equipment; 3) building contents. These components represent a great percentage of the capital investment in most commercial buildings (Whittaker and Soong 2003). Non-structural systems are very vulnerable to earthquake shaking, as observed from past damage reports, and high socio-economic losses and business interruption are related to their damage, especially for low seismic intensity motions. In addition to the potential risk to life-safety, their survival is essential to provide emergency services in the aftermath of an earthquake.

In the research field, more and more effort has been dedicated to better understand the seismic behaviour and damage states of non-structural elements, through laboratory testing, earthquake damage data and analytical and/or numerical investigations. With the aim of mitigating their risk of damage, recent research is moving towards the development of damage-control technologies for these building elements. Therefore, innovative solutions have been proposed for both vertical, e.g. cladding systems, infill walls, drywall partitions (Baird et al. 2011, 2013; Tasligedik et al. 2014; Tasligedik and Pampanin 2016), and horizontal, e.g. ceilings, non-structural components (Pourali

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et al. 2017), with the aim of improving their seismic performance, thus reducing the post-earthquake losses associated to non-structural damage.

Post-earthquake damage

A brief overview on the damage states affecting the non-structural elements as observed from past but more recent earthquakes (L'Aquila 2008, Darfield 2010, Christchurch 2011, Kaikoura 2016) is herein reported, taking into account the damage data found in De Sortis et al. (2009), Dhakal (2010), Baird et al. (2011), Baird and Ferner (2017) and focusing on architectural systems.

Damage to exterior enclosures

The main damage states affecting façade systems (Figure 1) are: 1) for *curtain walls*, typical damage consisted of falling glass for the presence of insufficient allowable movement of the panels and warping of the steel or aluminium frame and its total or partial disconnection from the structural system due to the presence of metal connections not appropriately designed; 2) for *cladding walls*: lightweight panels usually damaged for the absence of allowance to the relative structure-component movement and the damage consisted of cracking, tearing or disconnection of the panel; heavy panels, such as precast concrete systems, showed cracking, corner crushing especially due to pounding between panels, falling of bolts and panel disconnection, ejection or rupture of sealing joints; 3) for *infill walls*, heavy systems interacted with the primary structure leading to unexpected effects both at local and global level, while lightweight infill walls were usually damaged due to the absence of in-plane movement allowance.

Figure 1. Examples of damage to curtain walls (left), precast concrete cladding systems (centre) and masonry infill walls (right) from Baird et al. (2011).

Damage to partitions

During last earthquakes many buildings suffered moderate-to-extensive damage to partitions and high economic losses were associated to such type of non-structural elements.

Light partitions may be damaged as a result of in-plane or out-of-plane effects if not properly detailed and the most common observed damage were fastener damage at top and bottom connections, dislodgments of studs, linings cracked or detached from the framing, failure of anchorage between partition frame and structural members. Heavy partitions may affect and change the overall building response and can be both acceleration and displacement sensitive, falling for either in-plane or out-of-plane movements if not properly detailed. Masonry walls may crack and spall, creating debris which is particularly hazardous in stairwells and elevator shafts.

Damage to ceilings

Damage to ceiling systems represents one of the widespread consequences after earthquakes. Referring to suspended ceiling systems, typical damage included dislodging and breaking of tiles, failure of grid members, connections and perimeter angles, displacement incompatibilities, interaction with the structure or other systems.

Concerning ceiling systems directly applied to structural elements, if the finish material is not well anchored to the structure, this may pose a falling hazard. Typical damage is related to panel cracking or cracking located around the edges between ceiling and walls or the seismic joints.

Research motivation

In the past, earthquake engineers have mainly focused on improving the seismic performance of structural systems, therefore more comprehensive standards developed and expectations of advanced seismic behaviour increased. As either the earthquake engineering community or the public demand higher level of earthquake protection, it becomes fundamental to understand the seismic behaviour of non-structural elements and the related post-earthquake damage.

The damage of non-structural components during earthquakes may result in substantial economic losses and business interruption and may cause injuries or fatalities. Therefore, the study of non-structural seismic performance is becoming important and a great effort has been dedicated in the last decades to better understand the damage states of non-structural elements with the aim of developing innovative solutions able to mitigate their risk of damage.

In order to facilitate the quantification of damage levels as well as the proposal of efficient damage-resistant technologies, a collection of the key parameters (mechanisms and damage states) of each non-structural system is needed. As initial effort towards this objective, the paper presents a general overview on the seismic vulnerability of alternative systems, either based on traditional construction practice and on innovative solutions. Therefore, since many different systems are available worldwide, a collection of main literature references regarding the study of the seismic behaviour and damage states for facades, partitions and ceilings is presented. Finally, for the case of precast concrete cladding façades, numerical investigations are performed in order to define and compare the fragility functions of traditional and low-damage solutions.

Fragility study of non-structural components

The damage estimation of non-structural elements is an essential part of the performance and loss assessment of a building. Damage information can be defined from data gathered during earthquakes, laboratory experiments or numerical analyses on the components. The identification of the damage states as output from these studies allow the definition of the fragility curves, which represent the probability that a specific component response to various seismic excitations exceeds performance limit states. Non-structural elements can be sensitive to acceleration, inter-storey drift or both and the damage states related to these response parameters are converted into fragility curves to be used in probabilistic structural analyses.

Fragility functions can be built from data available in past researches, especially from experimental results which are one of the most reliable sources of data to evaluate the damage progress in a component subjected to defined loads. Obviously, fragility relations vary in function of the system details, i.e. for a glass façade it depends on the framing detailing, glass-to-frame clearance, system type, glass type, panel dimensions, glass thickness, but general considerations on the expected behavior of a system can be identified.

Regarding the facades, the following fragility relations can be considered:

- *Glazing systems*: The damage of a glass façade typically starts with the degradation of the gasket, representing a serviceability damage state, that allows air and water infiltrations. Thus, it can be observed initial glass cracking and crushing, still considered serviceability failures which produce air leakage, water infiltration, and other indirect damages that can increase the costs to building owners and occupants. Finally, there is glass fallout, an ultimate damage state representing a potential life safety hazard. A large database of fragility curves for different types of glazing systems in terms of inter-storey drift ratios can be found in the work of O'Brien et al. (2012) or in the FEMA P-58 (2012).
- *Cladding systems*: Depending on the system configuration, panel properties, type of connection and its design, different damage states can be identified. The performance levels for cladding panels can be classified as: 1) undamaged panels with no visible cracks, 2) minor cracking, 3) major cracking and crushing at connections. Regarding the connection system, different types of connector body (bearing, tie-back, slotted or dissipative) can be used, each of them characterized by proper damage states. As example, for threaded rod connections, the performance levels go from undamaged condition (pre-yield), to the connection yielding with no observable damage, to visible

cracking in connections, to severe cracking with loss of cross-sectional area, to rupture of rod and disconnection of panel (Baird et al. 2012).

- *Infill walls*: several researches focused the attention on the development of fragility curves from experimental data for masonry infill walls (such as Cardone and Perrone 2015), as a function of the peak inter-storey drift ratios for the in-plane seismic behaviour and peak floor acceleration for the out-of-plane response. Notwithstanding the definition of fragility curves is related to the masonry type and panel aspect ratio, the damage states are generally light cracking, extensive cracking, corner crushing, collapse.

Concerning drywall partitions, several studies on fragility curves are available for both steel and timber framed panels (e.g. Petrone et al. 2015). For gypsum walls, three main damage levels can be identified: 1) minor damage, to be repaired using just tape, mud and paint; 2) a second damage state requiring sections of gypsum to be cut out and replaced; 3) a third damage level corresponding to walls damaged beyond repair. As example, typical damage to timber framed panels are damage at fastener heads, cracking at wall openings, crushing and/or cracking at perimeter walls, cracking of the panel joints, local buckling of sheathing, and global buckling of sheathing.

In the case of ceiling systems, many experimental tests have been performed for suspended ceilings to determine the fragility of either the single components, including grid members -main tees and cross tees- as well as connections with cross tee connections, main tee splices and end fixing rivets (Dhakal et al. 2015), or the entire system (Badillo et al. 2007). Fragility functions on different types of suspended ceilings can be also found in FEMA P-58 2012.

Table 1 presents a description of the damage states required to build the fragility curves of some types of non-structural components. For each traditional system, the damage-mitigation strategy proposed by different authors is also introduced to highlight how these low-damage technologies, which rely upon connections detailed with relative movement between components and/or supplemental dissipation devices (Baird et al. 2013; Tasligedik et al. 2014; Tasligedik and Pampanin 2016; Pourali et al. 2017), aim to move the damage states towards higher seismic demand, thus reducing the expected losses associated to non-structural damage.

Non-structural element	**Type of system**	**Description of the system**	**Data for fragility curves**
Glazing façade: Spider glazing	Traditional (Sivanerupan et al. 2014)	12 mm toughened glass, 8 mm silicon sealant, Pinned X-type spider arms	DS1. Glass fallout (θ = 2.1%, σ = 0.5)
	Low-damage (Sivanerupan et al. 2014)	12 mm toughened glass, 8 mm silicon sealant, Fixed K-type spider arms with 7 mm horizontal and 17.5 mm vertical gaps	DS1. Gasket degradation (θ = 2.0%, σ = 0.5) DS2. Glass fallout (θ = 5.25%, σ = 0.5)
Cladding façade: Connection system for precast concrete panels	Traditional (Baird et al. 2014)	Threaded connection, Rod diameter 20 mm, Rod length 250 mm	DS1. Pre-yielding (θ = 0.2%, σ = 0.2) DS2. Post-yielding; visible cracking (θ = 0.5%, σ = 0.2) DS3. Severe cracking (θ = 1.0%, σ = 0.2) DS4. Rupture of rod (θ = 2.0%, σ = 0.2)
	Low-damage (Baird et al. 2013)	UFP connection, 120 x 8 mm steel plate	DS1. Pre-yielding (θ = 0.18%, σ = 0.2) DS2. Post-yielding; visible cracking (θ = 2.7%, σ = 0.2)
Infilled façade: Masonry infill walls	Traditional (Cardone and Perrone 2015)	Masonry infills with French window and partitions with door	DS1. Detachment of infill, Light diagonal cracking (θ = 0.15%, σ = 0.5) DS2. Extensive diagonal cracking

Infilled façade: Masonry infill walls	Traditional (Cardone and Perrone 2015)	Masonry infills with French window and partitions with door	(θ = 0.4%, σ = 0.5) DS3. Corner crushing and sliding of mortar joints (θ = 1.0%, σ = 0.4) DS4 Global collapse (in-plane) (θ = 1.75%, σ = 0.35)
	Low-damage (Tasligedik and Pampanin 2016)	Rocking walls with lateral gaps of 10 mm, 1.5 - 2 aspect ratio	DS1. Minor horizontal mortar cracking (θ = 1.5%, σ = 0.5) DS2. Light mortar cracking, minor toe-crushing (θ = 2.5%, σ = 0.5)
Drywall partition: Steel stud gypsum wall	Traditional (FEMA P-58 2012)	Gypsum with metal studs, Full height, Fixed below and above	DS1. Screws pop-out, minor cracking of wall board, warping or cracking of tape (θ = 0.21%, σ = 0.6) DS2. Moderate cracking or crushing of gypsum (θ = 0.71%, σ = 0.45) DS3. Significant cracking and/or crushing of gypsum, buckling of studs, tearing of tracks (θ = 1.2%, σ = 0.45)
	Low-damage (Tasligedik et al. 2014)	Gypsum with metal studs, 15 and 5 mm external and internal gaps, Gypsum boards attached only to the vertical studs	DS1. Minor plaster cracking (θ = 1.0%, σ = 0.4) DS2. Anchor pull out of the external studs (θ = 2.0%, σ = 0.4)
Suspended ceiling Suspended lay-in acoustic tile ceiling	Traditional (FEMA P-58 2012)	Area < 250 sf, Vertical hanging wires only	DS1. 5 % of tiles dislodge and fall (a = 0.9g, σ = 0.4) DS2. 30% of tiles dislodge and fall and t-bar grid damaged (a = 1.5g, σ = 0.4) DS3. Total collapse (a = 2.2g, σ = 0.4)
	Low-damage (Pourali et al. 2017)	Elastic acoustic isolation material into the lateral gap	DS1. 5 % of tiles dislodge and fall (a = 1.22g, σ = 0.4)

Table 1. Damage states of some traditional and low-damage non-structural components.

Risk assessment analysis of precast concrete claddings: traditional solution vs. low-damage solution

In this section of the paper a risk assessment analysis is presented in order to develop fragility functions for cladding panel connections. Referring to an initial work proposed by Diafeira et al. (2011) and implementing Incremental Dynamic Analyses, the probability of damage of cladding systems with traditional tie-back connections after earthquakes of different intensity is initially determined. Thus, the same investigation is performed considering innovative connections (U-Shape Flexural plates) to highlight the benefits of adopting damage-resistant technologies for this type of non-structural system.

Case-study structure

A multi-storey reinforced concrete case-study building with cladding panels as exterior enclosure is analyzed to develop the proposed fragility study. The structure has 5 storeys, plan dimension of 32 m x 18 m and inter-storey height of 3.8 m. The building use is commercial for the first two floors, residential for the other two, while the top floor is a roof. The monolithic structural skeleton consists of seismic resistant four-bay frames in one direction and shear walls in the orthogonal direction (Figure 2- left), whereas precast concrete claddings cover the external façade of both building directions. The building is located in Norcia, Italy (PGAs of 0.341 g, Soil type C) and has been designed at the Ultimate Limit State (ULS) level (475 years return period earthquake for an Importance Class 2) following the Direct Displacement-Based Design procedure (Priestley et al. 2007; Pampanin et al. 2010).

The precast concrete panels are 100 mm thick and include a central opening. The claddings are composed by dual-panel systems of 8 m total length in the frame direction (Figure 2- centre) while mono-panel systems of 6 m in the wall direction. These panels are connected to the structural skeleton using two bearing (fixed) connections at the bottom of the panel while different types of connections are considered on the top (Figure 2- right): two traditional tie-back and/or slotted connections, designed considering a suggested drift of 0.2% (Baird et al. 2011), or dissipative U-Shaped Flexural Plate (UFP) connections, designed referring to Baird et al. 2013.

The longitudinal seismic frame of the structure, with first mode period of 0.86 s, has been chosen to implement the proposed investigation.

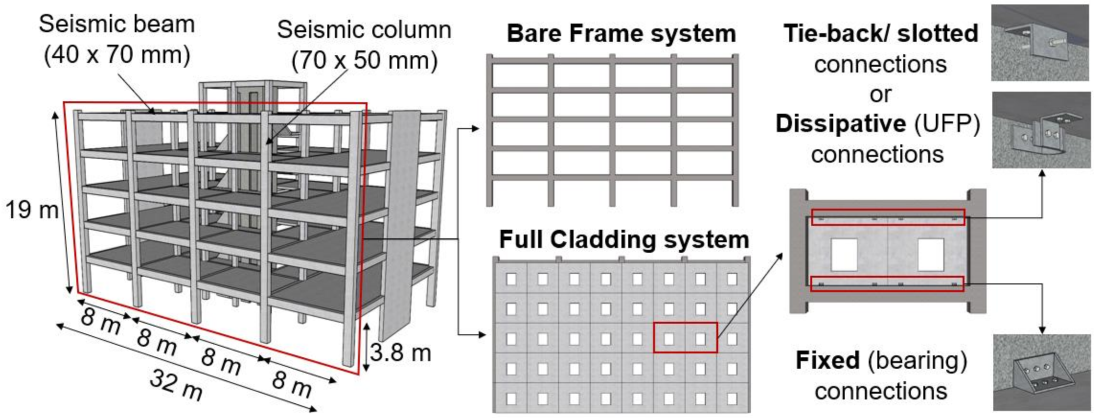

Figure 2. Case-study Building: global view and frame dimensions (left); building systems considered for the analyses (centre); precast concrete cladding panel configurations (right).

Initial numerical investigation

Initial numerical non-linear static analyses have been carried out using Ruaumoko 2D software (Carr 2003) and a lumped-plasticity approach with the aim of validating the system modelling, also highlighting the influence of the non-structural systems in the global structural behaviour.

The structural skeleton has been modelled by mono-dimensional elastic elements with plastic hinge regions at the end sections (Giberson elements) where the inelasticity is represented by appropriate moment-curvature relationships and stiffness-degrading hysteresis rules (i.e. Takeda). The precast concrete panels have been modelled through an equivalent spring model which consists of a single linear spring representing the cladding panel and top and bottom connections described respectively by horizontal springs (tie-back or UFP - Bounded Ramberg Osgood - Figure 3, right) or dash-pots (slotted - Coulomb Dash-pot) and rigid links (bearing). The in-plane stiffness of the panel with a central opening as well as the properties of the connection elements have been calibrated using the formulas proposed by Baird (2014). The rotations and axial displacements of these elements were restrained so that they can only deform horizontally.

Non-linear static pushover and push-pull analyses have been initially performed, considering different connections on the top of the cladding panel: 1) tie-back connections made of long (250 mm) or short (50 mm) threader rods of 20 mm diameter; 2) slotted connections with 150 mm slot length; 3) UFP connections composed by a 140 x 10 mm steel plate.

The obtained push-over curves are presented in Figure 3 (left) and compared to the capacity curve of the bare frame system. This figure shows how the influence of such connection systems in the global response is limited. In fact, it can be observed how the stiffness and strength of the structure increase by a value of 3% for both the tie-back connections with long rods and slotted connections, 16% for the tie-back connections with short rods and 12% for the UFP connections.

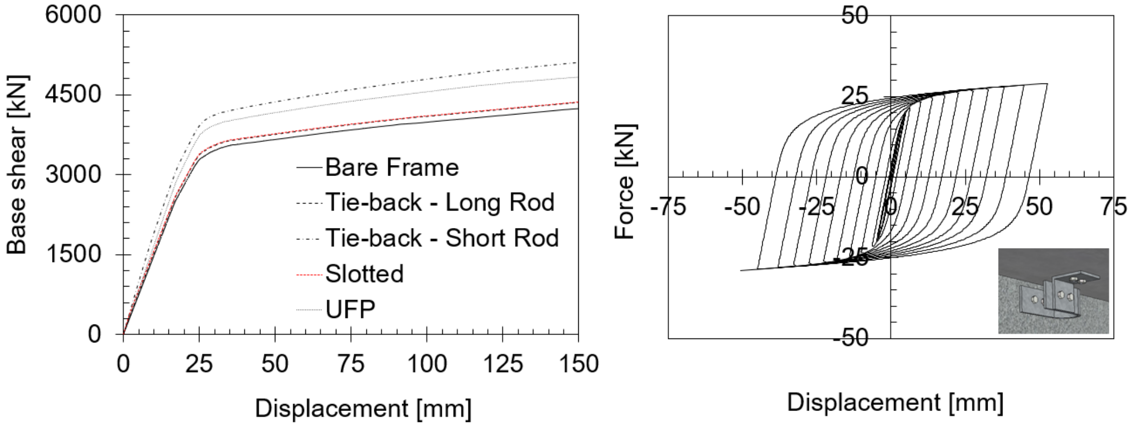

Figure 3. Numerical push-over curves (left) and hysteresis behaviour of UFP connection (right).

Notwithstanding the interaction with the structural system is very reduced, tie-back connections are expected to fail when a drift of 1.5-2% is reached. For slotted connections, if the displacement demand exceeds the slot capacity, the force transferred to the cladding system is much greater than the friction force alone and the connection becomes a fixed end threaded rod connection that is expected to fail at a certain point. When these connection fail, the heavy precast concrete panels may detach and this damage condition may cause risk of hazard to human life, apart from the related substantial economic losses. Regarding the UFP connections, these dissipative connections reduce the demand on the panels, thus the consequent damage, and are not expected to fail due to the achievement of a maximum displacement but for fatigue criteria (Kelly et al. 1972), that is in this case a minimum of 150 cycles (at the maximum stroke).

Incremental Dynamic Analysis

The probability of the cladding connections damage can be determined through the implementation of a seismic risk assessment. The reaching/overcoming of damage states conditions and the related fragility functions can be obtained through the implementation of the Incremental Dynamic Analysis (IDA) procedure proposed by Vamvatsikos and Cornell (2002). The connection behaviour can be defined from available experimental tests and expressed in terms of “connection drift”, which represents the relative displacement of the connection divided by the inter-storey height. Thus, the probability of reaching this Engineering Demand Parameter (EDP) as a function of an appropriate Intensity Measure (IM) can be found.

The IDA investigation has been implemented considering a suite of 15 ground motion records representative of events likely to cause from moderate to severe shaking motions in the Norcia area. The number of events selected are enough to provide sufficient accuracy in the estimation of seismic demands for mid-rise buildings, assuming an efficient IM, like the spectral acceleration $S_a(T_1,5\%)$ (Shome and Cornell 2002). In fact, the IDA curves are obtained scaling the 15 accelerograms from 0.2g to 2g with a step of 0.2g in relation to their spectral acceleration $S_a(T_1,5\%)$ and a total of 150 analyses have been performed for every connection analysed.

Development of fragility curves

The fragility study is carried out referring to the case of long threaded rod tie-back connections, having better performance than the short rod solution whilst are better numerically described than the slotted connections, whose dash-pot model needs to be adjusted to take into account the stiffness increase due to the achievement of the total slot length.

Taking into account that the performance of non-structural elements also depends on the response of the primary structure, the fragility curves have been prescribed to consider the occurrence building global collapse. Therefore, referring to the “total probability” (Jalayer 2003) both the conditions of building collapse, assumed as the achievement of 4% drift as for the FEMA

356 (2000) recommendations, and no-collapse are considered such as the functions describing the probability of occurrence building collapse/irreparability can be defined using a log-normal distribution (Figure 4 - left). Based on this result, the fragility curves associated to the first tie-back connection reaching the yielding can be built considering the damage states presented in the previous Table 1 (Figure 4 - right). These fragility functions are compared to the ones related to the UFP connection adopted as alternative solution for the same cladding component.

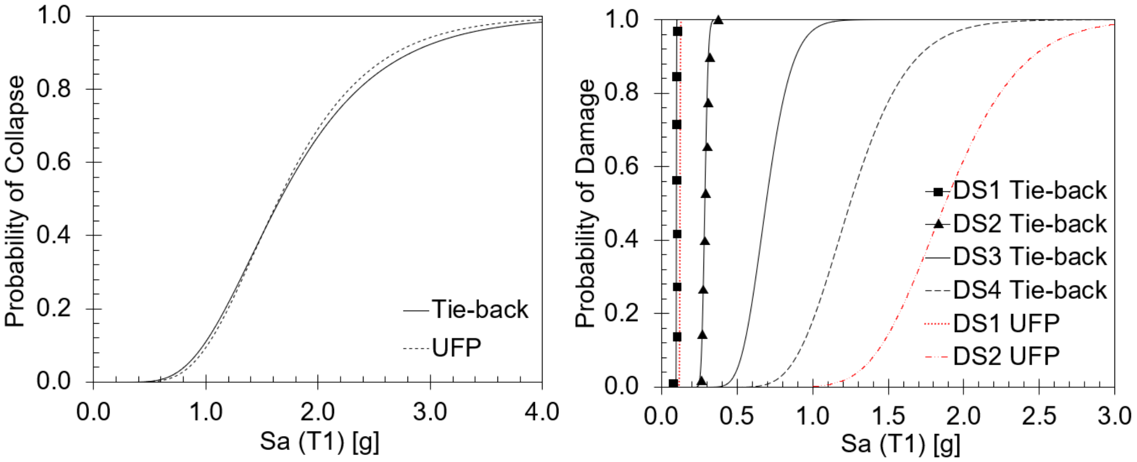

Figure 4. Probability of occurrence collapse/irreparability of the Full Cladding system with tie-back or UFP connections (left); fragility curves of the traditional or low-damage connection of the lateral cladding on the third building floor (first connection which yields).

The previous graphs show how, notwithstanding the probability of collapse/irreparability of the building is very similar introducing tie-back or UFP connections on the top of cladding panels, the beneficial effects of the low-damage system in the fragility curves are evident. As previously described, the UFP connections fail for fatigue criteria but a damage state (DS2) related to the exceedance of the designed slot allowance of the connection can be considered. Both types of connections are designed to yield at 0.2% drift, but it can be noticed how for example at 1.5g of S_a (T_1) the probability of being in DS4 for the tie-back connection is around 80% while for the UFP connection the probability of being in DS2 is around 10%.

The damage probabilities from the previous graph can be expressed in function of the annual frequency or return period of the earthquakes using the formula proposed by Maniyar et al. 2009. In this way, the probability of defined damage conditions not being exceeded for seismic demand of various annual probabilities can be determined (Figure 5).

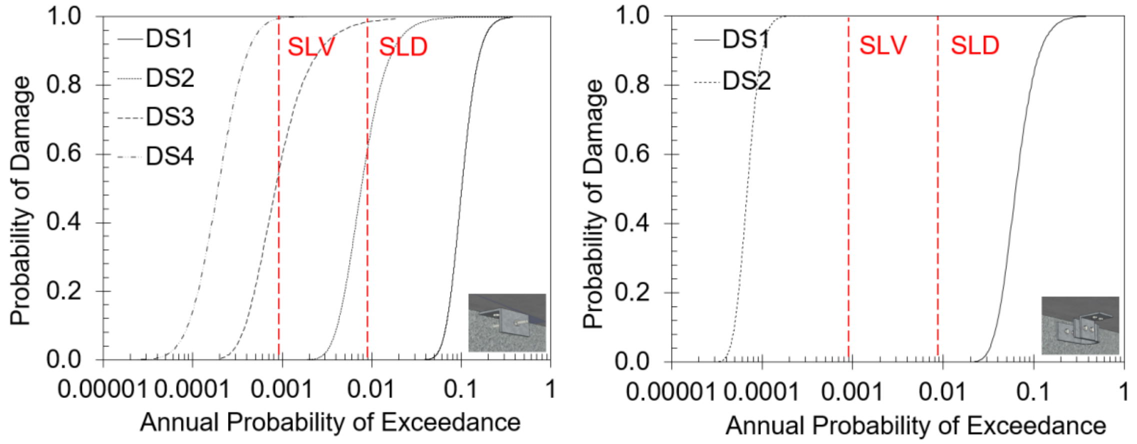

Figure 5. Probability of damage in relation of the annual probability of exceedance for both the tie-back connection (left) and UFP connection (right).

Referring to the seismic intensity levels of Damage Control (SLD, T_R = 50 years), it can be observed how for the tie-back connection the probability of the damage states not being exceeded is around 60% for DS2, 95% for DS3 and 100% for DS4, while for the UFP connections the probability of not being exceeded is 100% for DS2. In the case of Life-Safety condition (SLV, T_R = 475 years) for the UFP connections the probability remains 100% whilst for the tie-back connections the probability for DS3 of not being exceeded becomes approximately 60%. This highlights how the low-damage connections have a very high probability to not being damaged compared to the traditional system, in addition to the fact that these solutions lead to very reduced post-earthquake damage to the cladding panels, thus limited associated losses, because of their capability of dissipating the seismic energy.

Conclusions

The study of the seismic behavior of non-structural elements is becoming fundamental for the assessment/design of buildings, considering the influence that they have in the post-earthquake losses. Consequently, it is important to understand the potential damage states of these components and find the better and not-expensive way to improve their seismic performance.

Highlighting this concept, the research tries, as initial study, to provide information on the fragility specifications which characterize the behaviour of different types of non-structural components (facades, partitions, ceilings).

The seismic performance of these traditional construction practice systems can be improved introducing innovative low-damage solutions, that started to be developed and studied in recent years. These damage-mitigation solutions allow a reduction of the expected economic losses, as proved by Bianchi et al. (2018, 2019), due to the achievement of the corresponding damage states for very high displacements or accelerations, depending on the system sensitivity. Therefore, with the aim of proving this sentence, a qualitative risk assessment investigation is performed to show the convenience of implementing this new technology for the case, as example in the research, of heavy precast concrete cladding panels. The results of the implemented investigation highlight that, notwithstanding both the traditional and innovative connections are designed to yield at the same drift level, the collapse of the traditional system is reached for very lower seismic intensities compared to the innovative solution.

However, more investigations are needed to define a database with all the fragility information of different non-structural element typologies, at least as general considerations, also taking into account the variation of the system details and construction practice related to the country where these components are realized. Further studies are also required to build fragility curves and compare solutions considering not only the in-plane failures related to drift ratios but also including the out-of-plane behavior of these elements due to acceleration responses.

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