



UNIVERSITÀ
DEGLI STUDI
DI PADOVA

Head Office: Università degli Studi di Padova

Department of Civil, Environmental and Architectural Engineering

Ph.D. COURSE IN: Civil and Environmental Engineering Sciences
SERIES XXX

SEISMIC ASSESSMENT OF SUSPENDED CEILINGS THROUGH CYCLIC QUASI-STATIC TESTS

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*To Sara,
my suspended ceilings colleague
and friend*

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Abstract

The purpose of this work is the evaluation of the seismic behaviour of suspended ceilings by means of a combined experimental and numerical approach.

As concerning the experimental aspects, nowadays the most common experimental procedure applied to suspended ceilings worldwide regards shake-table tests, with a protocol defined to certify the ceilings for a determined seismic level. This methodology has some shortcomings, including the high cost and the influence of the input chosen on the experimental results. In fact, these tests have the aim to certify the product rather than providing mechanical characteristics of the specimen tested. Moreover, the results obtained with the certification process cannot be extended to similar products or geographic zones with different seismic risk. To overcome these limitations, an innovative experimental setup for monotonic and cyclic testing of suspended ceiling systems was designed, realized and applied.

In order to have a complete characterization of suspended ceilings, an initial experimental campaign on inner joints was realized. These components, in fact, performed poorly in past earthquakes. In detail, 'standard' joints were compared to 'seismic' joints, specifically designed to resist to earthquake motion.

Real-scale tests were then performed on the most common T-Grid suspended ceilings and other two typologies of metal ceilings with different structure. Moreover, dry-wall suspended ceilings with single and double plasterboard were tested. For each typology one monotonic and one cyclic quasi-static test were performed. Monotonic tests have the aim to identify the yielding parameters and the damage mechanisms and cyclic tests are performed according to the protocol described in FEMA 461 for cyclic quasi-static tests of non-structural components. The results allow to define the performance of suspended ceilings and to elaborate their capacity curves.

As concerning the numerical part of the work, a numerical cascading approach was applied to study the uncoupled behaviour of suspended ceilings installed at different levels of buildings. Results from experimental campaign were used as input data for the numerical analyses. Elastic linear time history analyses were performed on multi-story buildings with different vibration periods and the elastic floor response spectra were defined. Capacity curves defined experimentally and floor spectra were plotted in an ADRS (Acceleration Displacement Response Spectrum) domain in order to assess the seismic demand in terms of acceleration and displacement of suspended ceilings compared to their capacities. Dynamic analyses of suspended ceilings were conducted both in linear and non linear hypothesis and the results compared in order to assess the effectiveness of standard linear, or equivalent linear, static calculations.

Sommario

Lo scopo della tesi è la valutazione del comportamento sismico di controsoffitti, tramite prove cicliche quasi statiche. La tipologia di prove più comune ad oggi, infatti, riguarda prove su tavole vibrante con un protocollo definito per certificare il prodotto per una certa azione sismica. Queste prove presentano varie limitazioni, tra cui il costo elevato e la stretta correlazione tra risultato e input scelto. Le prove infatti non hanno specifico scopo di ricerca se non l'obiettivo di certificare un prodotto, non forniscono informazioni sulle prestazioni meccaniche dei componenti testati e non permettono di estendere i risultati ottenuti ne su prodotti simili ne in zone geografiche con diverso rischio sismico. È stato quindi progettato un setup di prova innovativo in grado di realizzare prove monotone e cicliche quasi statiche su controsoffitti. Questa tipologia di prove permette di superare le limitazioni dell'attuale procedura sperimentale.

Al fine di ottenere una caratterizzazione completa dei controsoffitti, sono stati testati i giunti interni, questi componenti infatti sono risultati danneggiati in seguito a eventi sismici. In particolare, sono stati testati sia giunti 'standard' che giunti 'antisismici', facenti parte di una particolare linea progettata per resistere all'azione sismica.

Sono stati testati a grandezza reale sia controsoffitti con struttura a T (che rappresentano la tipologia più diffusa globalmente), che altri due controsoffitti con diversa sottostruttura metallica, infine le prove hanno riguardato anche controsoffitti con pannelli continui in cartongesso. Per ogni tipologia sono stati eseguite una prova monotona, al fine di individuare i parametri di snervamento e il meccanismo di rottura, e una prova ciclica, seguendo il protocollo indicato nelle FEMA 461 per prove cicliche quasi statiche per componenti non strutturali. I risultati ottenuti hanno permesso di definire la prestazione degli elementi testati e di elaborarne la curva di capacità. Tramite approccio numerico "a cascata", che permette di eseguire uno studio disaccoppiato dei due elementi, è stato possibile studiare il comportamento dei controsoffitti installati a diversi piani. Sono state realizzate analisi time-history lineari elastiche su edifici multi-piano con diverso periodo di vibrazione e sono stati ricavati gli spettri di risposta al piano. Le curve di capacità dei controsoffitti, definite sperimentalmente, e gli spettri al piano sono stati definiti in un dominio ADRS (Acceleration Displacement Response Domain) al fine di valutare la domanda sismica in termini di spostamento e accelerazione in funzione della capacità dei controsoffitti.

Introduction

The attention regarding the seismic behaviour of non-structural components has increased over last decades in consequence to the damages observed during and after earthquakes worldwide. In fact, damage to non-structural components can severely affect the functionality of buildings and has a great impact on the total economic losses caused by the seismic event. It can also result in injuries or deaths. Another critical issue is that such damage can also occur at low seismic intensity, for which structural elements are expected to remain in their elastic domain.

Among non-structural components, ceiling systems have great relevance due to their presence in commercial, residential and public use buildings. Suspended ceilings are generally used to conceal the building services (Griffin and Tong, 1992) and their use is widespread in both publicly- and privately-owned buildings.

In consequence of the raising interest and requests from customers, ceiling manufacturers worldwide developed their own seismic solution. Only in United States, specific Standards have been released, leading to uniformity of products and seismic design approach. Despite this, the overall impression emerging from the State of the art is inconsistency and inefficiency of the actual technique to realize seismic resistant ceilings and inaccuracy of the most common testing methodology, whose aim is to certify the product for a certain seismic level rather than investigate deeply its behaviour. Moreover, tests are mainly conducted on suspended ceilings produced in United States, while the products manufactured in Europe haven't mostly been tested yet. Finally, there is a discrepancy between the accurate numerical models available in Literature and the simplified analytical formulations proposed by Codes worldwide. Numerical models are too detailed and complex to be used by designers or engineers. Otherwise, Codes don't present a specific formulation for suspended ceilings rather than non-structural components and are proved to be inaccurate for computing the seismic force on suspended ceilings.

The aim of this thesis is to define a methodology for the seismic assessment of suspended ceiling systems, that will be validated through testing of different typologies of suspended ceilings.

The work started with an accurate study of the state of the art, developed in Chapter 1: it comprehends a detailed report of failures of suspended ceilings after earthquakes, a synthesis of the experimental campaigns developed worldwide and numerical and analytical models of suspended ceilings. Moreover, the analysis methods to compute the seismic force on non-structural components are presented, including formulations proposed by Codes. The main findings are presented and the limitations or lacks in the knowledge are also outlined.

The experimental characterization of suspended ceilings starts in Chapter 2 with the assessment of inner joints, since they are reported as one of the weakest components of suspended ceilings. Moreover, standard joints are compared to 'seismic' joints, designed by the manufacturer to be part of a seismic resistant technology related to T-Grid ceilings.

The shortcomings outlined from the state of the art regarding the shake-table tests on real-scale suspended ceilings suggested the need of changes in the actual experimental methodology. Therefore, an innovative approach that develops cyclic quasi-static tests on suspended ceilings is presented in Chapter 3. These tests are a common procedure for the evaluation of the seismic behaviour of walls in their plane but the idea of performing static or cyclic tests on full-scale suspended ceilings is innovative and hasn't been applied yet to obtain their mechanical parameters. To realize the tests a setup able to reproduce the real boundary conditions and ceilings installation is built. For each specimen one monotonic and one cyclic quasi-static tests are performed. With the aim of performing a comprehensive assessment, different typologies of suspended ceilings are tested. The most common

T-Grid suspended ceilings are tested with both metal and gypsum tiles. Moreover, other two typologies of metal suspended ceilings are tested: their behaviour will be compared to that of T-Grid ceilings. Dry-wall suspended ceilings are also included in the experimental campaign and tests include specimens with single or double layer of plasterboards. Finally, the results were processed with the aim of defining mechanical parameters and capacity curves of suspended ceilings.

A simplified numerical methodology was applied in order to study the behaviour of suspended ceilings installed at different levels and it is presented in Chapter 4. Linear time history analyses were performed on multi-story buildings and elastic floor spectra were obtained for each floor. Floor spectra and capacity curves were defined in an Acceleration-Displacement Response Spectrum (ADSR) domain in order to assess the seismic demand in terms of acceleration and displacement on suspended ceilings compared to their capacities.

1 State of the art

This chapter has the aim of presenting the current state of the art in the seismic performance assessment of suspended ceiling systems and in existing methods of seismic analysis and design.

First, suspended ceilings components are defined and the main seismic construction methods offered by guidelines are described. An overview on the main failures of ceiling systems during past earthquakes is presented to classify the possible failure mechanisms, highlighting the weakest components and bad installation methods.

In recent years many experimental campaigns on full-scale ceiling systems were carried out, and also the most vulnerable suspended ceiling components were tested. The main limitations of the actual experimental methodology are explained, in order to outline the improvements required and the reasons of the realization of the innovative experimental setup described in this work. Moreover, the results presented in literature allow to compare the seismic response of different construction technologies. Numerical models of suspended ceilings presented in Literature are described and critically analysed.

As simplified structural analyses cannot describe properly the behaviour of suspended ceilings under seismic action, various methods are proposed to estimate the seismic response parameters of suspended ceilings, derived from empirical considerations or structural dynamic principles. These methods are briefly described and their use and limitations are discussed. Moreover, suspended ceiling systems are not considered specifically in the seismic sections of building Codes, rather non-structural components as a whole. Then, the formulations presented in the European and US Codes for computing the seismic force on non-structural components are described and discussed.

1.1 Suspended ceiling components and typical installation worldwide

Suspended ceiling systems are complex assemblies and their configuration is variable depending on system design. A first classification can divide metal ceilings from continuous dry-wall suspended ceilings. Among metal ceilings, the most common system is T-Grid suspended ceilings shown in Fig. 1. The main structure is a metal grid, made of T-shape main and cross runners, positioned perpendicularly to each other. Between them tiles are suspended, manufactured from metal or fibrous materials. To connect the grid to the ceiling, suspensions systems are positioned with regular spacing, they can be wires, rods or other profiles. Finally, perimeter trims are located at the intersection of the suspended ceiling with the wall, so that runners can be either rigidly connected or simply supported to the trims. When cut panels are located at the perimeter, a perimeter wedge is positioned on the tile to keep it in place if the tile has been cut to size.

Dry-wall suspended ceilings have a metal structure realized with channel or metal profiles to which the plasterboards are connected with screws. As it can be seen in Fig. 1, primary channel is defined the profile positioned at the higher level, while the secondary channel is placed perpendicular to the previous one and at a lower level, so that plasterboards are screwed to it. Details of perimeter elements or hanger wires are similar to T-Grid suspended ceilings. Dry-wall suspended ceilings can be installed without the primary grid and are named single-frame ceilings, while double-frame ceilings have both primary and secondary grid.

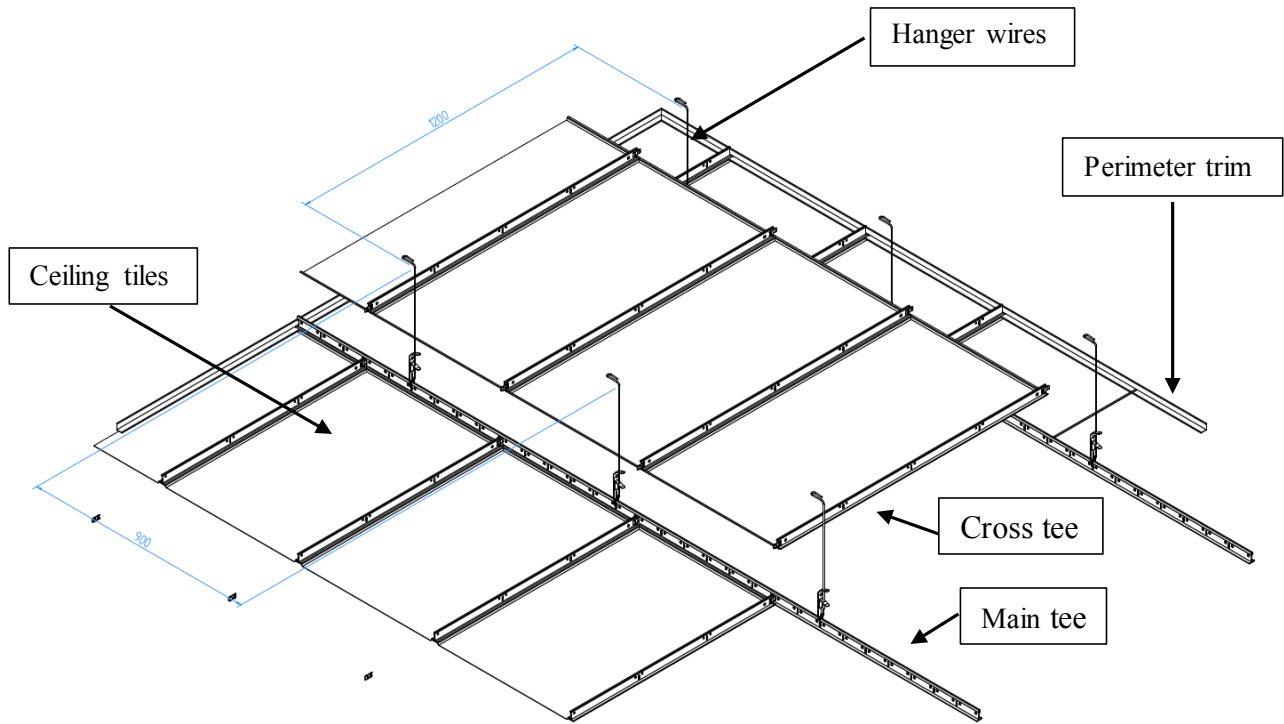


Fig. 1: Example of a typical T-Grid suspended ceiling (courtesy of Atena spa)

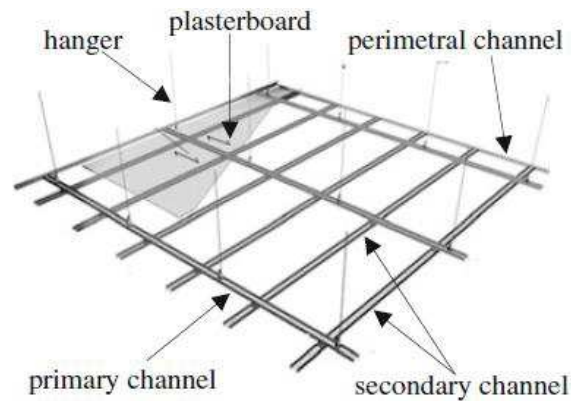


Fig. 2: Example of a typical dry-wall suspended ceiling (Magliulo et al., 2012)

Regarding the seismic resistance of suspended ceilings, some guidelines were published in USA describing suggested construction methods of suspended ceilings that comply to seismic requirements.

FEMA E-74 (2015) is a practical guide whose aim is explaining in simple terms damages caused by earthquakes to non-structural components and providing methods for reducing the potential risk. A section is dedicated to suspended ceilings and installation examples are reported. The interventions described are referring to ASTM E580, that is the most update Standard regarding the installation of suspended ceilings in seismic areas.

ASTM E580 “Standard practice for installation of ceiling suspension systems for acoustical tile and lay-in panels in areas subject to earthquake ground motions” (2016) replaces previous Cisca Standards (2004) and includes prescriptive recommendations on installation methods to be used for suspended ceilings in areas subjected to seismic events.

For suspended ceilings installed in buildings with moderate seismic vulnerability and maximum average weight of 12 N/m^2 , the aim is providing a fully-floating ceiling, meaning an unrestrained ceiling that can accommodate the movements of the building. The perimeter elements must provide

at least 22-mm support ledge (see Fig. 3 a). Moreover, runners must have a minimum 10-mm clearance from the wall and elements to prevent spreading must be installed within 200 mm of each wall (see Fig. 3 a). Finally, permanent runner end attachment (such as pop rivets) are not allowed. For suspended ceilings installed in buildings with high or very high seismic vulnerability, or with average weight higher than 12 N/m^2 , the aim is providing a restrained ceiling through either connection to the perimeter wall, or through bracing either rigid or non-rigid. The perimeter closure angles or channels must provide at least 50-mm support ledge and runner ends must be attached to the perimeter on two adjacent walls with a clearance of 19 mm between the runner ends and the perimeter members on the two opposite walls (see Fig. 3 c). Moreover, terminal end of runners must be supported independently with a 2.7 mm wire or other support at a maximum distance of 200 mm and elements to prevent spreading must be installed within 200 mm of each wall (see Fig. 3 c). Lateral bracing (also indicated as ‘seismic bracing’) is required only for ceiling areas higher than 92.9 m^2 (Fig. 3 b). It must be realized with four 2.70 mm wires secured to the main runner within 50 mm of the cross runner intersection and splayed 90° from each other at an angle not exceeding 45° from the plane of the ceiling. A strut connected to the main runner at the location of the bracing wires must be fastened to the structure above. The strut is generally named “compression post” and must resist vertical forces induced by the bracing wires. These horizontal restraints must be placed 3600 mm on centre in both directions with the first restraint within 1800 mm from each wall. Finally, only heavy-duty main tees can be used rather than intermediate duty tees. Duty rating is related to the measure of applied uniform loading to a 1.2-m simply supported runner that causes a deflection of $L/360$ (3.5 mm), heavy- and intermediate-duty runners sustain load greater than 0.8 kN/m^2 and 0.5 kN/m^2 , respectively.

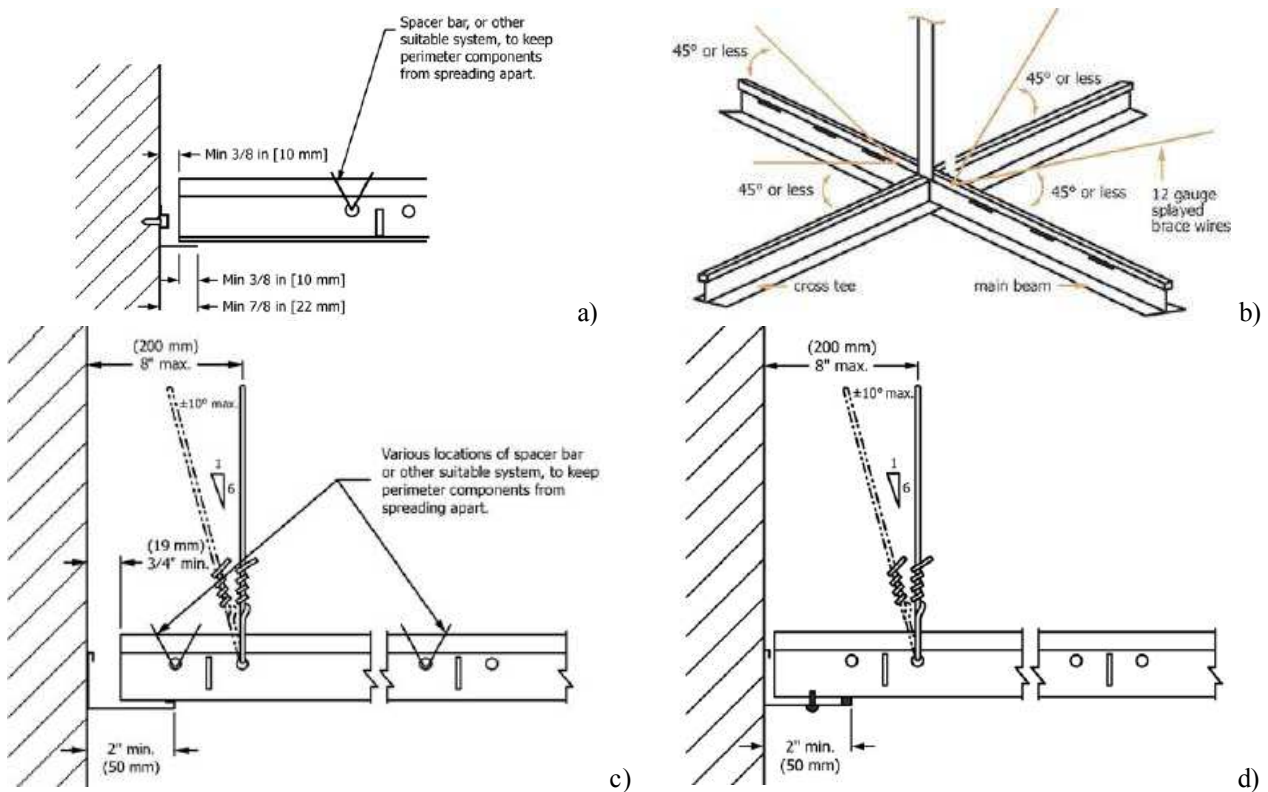


Fig. 3: ASTM E580 prescriptions for suspended ceilings. a) detail of connections at perimeter for suspended ceilings installed in buildings with moderate seismic vulnerability; b) lateral bracing; c) detail of free connection and d) fixed connection at perimeter for suspended ceilings installed in buildings with high or very high seismic vulnerability.

For all suspended ceilings, 2.7 mm galvanized wires are suggested as suspension systems and technical details on spacing and correct installation are also provided. Moreover, rules regarding the installation of lighting systems, services and elements penetrating the ceilings are indicated. Finally, for suspended ceilings wider than 232 m² and installed in building with high or very high seismic vulnerability, prescriptions on the realization of separation joints or full height partitions are reported. Dry-wall suspended ceilings are exempted from these prescriptions.

The International Building Code (IBC 2006) allow the use of alternative methods rather than the techniques previously described if the code official finds that they comply building regulations with the support of research reports from approved sources. The code official can also require tests as evidences of compliance and they should be performed with recognized and accepted testing procedures. Some perimeter elements were presented in the American market, in substitution to the ASTM E-580 requirements regarding the connections at perimeter (Fig. 4). In particular, they act as retainers for the runners, permit to realize fixed connections and allow to reduce the edge of the perimeter trim to 22mm-length.

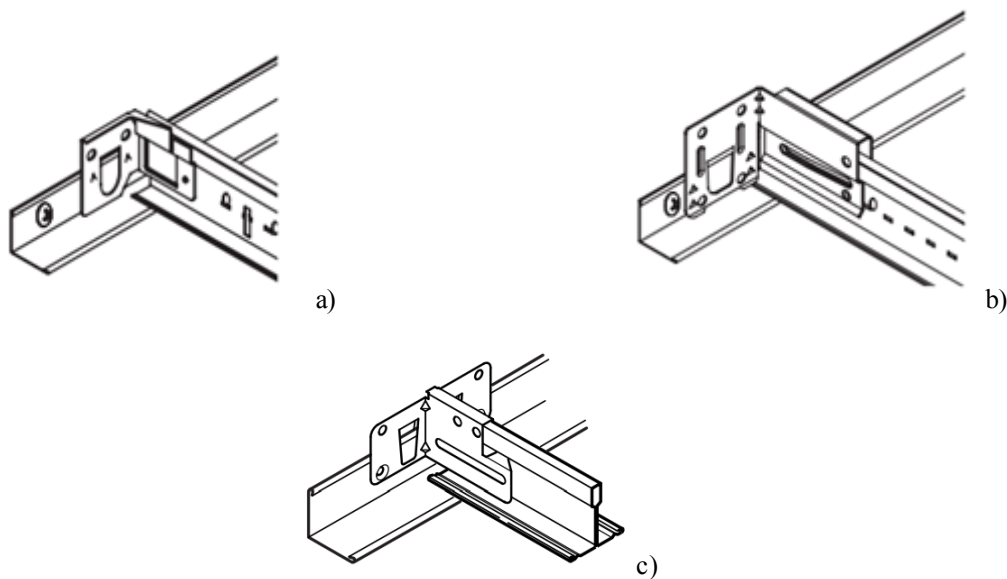


Fig. 4: a) Beam End Retaining Clip and b) Beam End Retaining Clip 2 by Armstrong; c) ACM7 Seismic clip by USG.

Other manufacturers settled worldwide patented elements to be part of the suspended ceilings systems for increasing their seismic resistance. Outside USA, the ASTM E580 is not prescriptive and is considered only as a guideline. The lack of rules regarding technical details of seismic resistant suspended ceilings allows manufacturers to create innovative products easily and a variety of seismic solutions can be found. The concept of seismic resistant suspended ceilings is similar to ASTM E580: lateral bracings connect main runners to the structure above and runners are fixed to the perimeter trim on two adjacent sides. Every manufacturer has its own installation rules, generally coming from common sense or experimental activity internal at the company. Some examples of commercial seismic solutions for suspended ceilings are presented in this work. A detailed description of them and their seismic behaviour assessment will be provided in next chapters.

1.2 Seismic damage of suspended ceilings

The damage conditions of suspended ceilings after an earthquake is the most evident and simple way to observe the critical aspects of suspended ceilings affected by earthquake motion. This can be a useful source of information to address a detailed study on suspended ceilings or their components. Conversely, it has been used in last years as the starting point for an empirical approach (Filiatrault and Sullivan, 2014), that means taking advantage of natural disasters as a test bench for the performance of suspended ceilings and using only intuition to design new products.

In this section, earthquake damage experienced by suspended ceilings are classified according to the suspended ceiling components affected by the damage.

1.2.1 Dislodging of panels

The action of the vertical component of seismic acceleration can induce dislodging of panels from their original position. If panels remain on grid, they can be easily repaired, and this is associated with a low damage level. To avoid these phenomena, retainer clips can be installed. In Fig. 5 a), dislodging of panels located at the perimeter is reported. In Fig. 5 b) and c) panels slightly moved from their original position.



a)



b)



c)

Fig. 5: a): Damage from September 2010 Darfield earthquake, photos by Hush Interior Ltd. (MacRae et al., 2011); b) Damage from 22nd February 2011 Christchurch earthquake. (Dhakal et al., 2011); c): Damage reported in 2008 in Northern Chino Hills Southern California. (Glasgow et al., 2010).

1.2.2 Fall of panels

If the dislodged panels are subjected to repetitive movements, they can fall down. The number of panels fallen from the suspended ceiling is a key value for the evaluation of the damage state. In fact, as the number of fallen panels increases, the risk of injuries or deaths becomes higher. Moreover, a

wide loss of panels causes the closure of the building for reparations. Consequently, this damage mechanism can cause economic losses, meaning of reparation cost and downtime. In Fig. 6 a), c), d) and g) an extensive collapse of panels located at perimeter is reported. In Fig. 6 b) and again in Fig. 6 d), few panels falls down separately in the internal part of the suspended ceiling. Finally, a wide damage is highlighted in Fig. 6 d) and e), where the fall of adjacent panels causes the collapse of large area of suspended ceiling.



a)



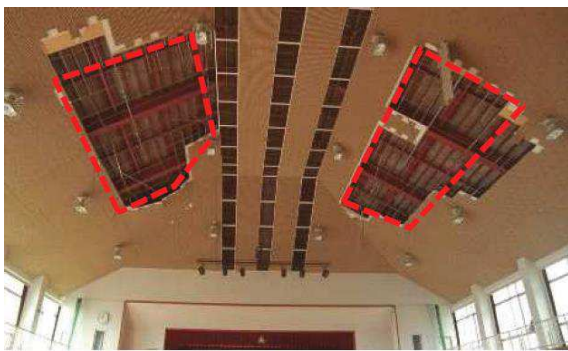
b)



c)



d)



e)



f)



g)

Fig. 6 a): Damage from the Niigata Earthquake in 2007. (Gilani et al., 2008); b) Damage reported in 2010 in Christchurch, New Zealand. (Gilani et al., 2014); c) Damage reported in 2007 from Niigata Japan Earthquake. (Glasgow et al., 2010); d) Damage reported in 2008 from Sichuan China Earthquake (Glasgow et al., 2010); e) Damage reported from the 2001 Geiyo Earthquake, Japan (Watakabe et al., 2012); f) Damage reported from the 2011 Great East Japan Earthquake (Watakabe et al., 2012); g) Damage reported from the 2010 Eureka earthquake, (Soroushian et al., 2015c)

1.2.3 Perimeter failure

Suspended ceilings can be constrained on two consecutive sides with elements that connect the grid to the perimeter trim (i.e. pop-rivets, screws, seismic clips). Perimeter failure is defined as the failure of these elements or the dislodging of tees from the perimeter trim when not mechanical constrained. This damage can induce dislodging or fall of perimeter panels, as represented in Fig. 7 a), d) and e). Another consequence can be the pounding of tees against perimeter trim, with damage located at walls or dislodging of tees (Fig. 7 c). A detail of damage at riveted connections is reported in Fig. 7 b).



a)



b)



c)



d)



e)



f)

Fig. 7 a): Damage reported from L'Aquila earthquake in 2009 (Magliulo, 2009); b): Damage reported after New Zealand earthquakes in 2010 and 2011 (Dhakal et al., 2013); c) and d): Damage reported from the 22nd February 2011 Christchurch earthquake (Dhakal et al., 2011); e): Damage reported after Napa (CA) earthquake in 2014 (Gilani et al., 2015); f): Damage reported after the 2010 Baja earthquake (Gilani and Takhirov, 2011).

1.2.4 Connection failure

Main tees are connected each other or to cross tees with typical joints. The failure of these joints cause connection failure, i.e. cross tees disconnecting from main tees or main tees separating in the point of connection. This damage can be caused by buckling of joints or web tearing of the perpendicular runner to which the joint is located. A detail of the joint damage is reported in Fig. 8 a), while in Fig. 8 b) runners in one side detached from the perpendicular tee, meaning that the stress was concentrated only in one side of the joint.



a)



b)

Fig. 8 a): Damage reported after the 22nd February 2011 Christchurch earthquake (Dhakal et al., 2011); b): Damage reported after New Zealand earthquakes in 2010 and 2011. (Dhakal et al., 2013).

1.2.5 Grid failure

Twisting, buckling, rupture or collapse of grid members is generally defined as grid failure and is related to the most extensive damage levels since it affects the metal skeleton of the suspended

ceilings. An extensive grid failure is indicated in Fig. 9 a), while in Fig. 9 b) and c) complete tearing of the runner's section is reported. Finally in Fig. 9 c), twisting of the tee is reported.

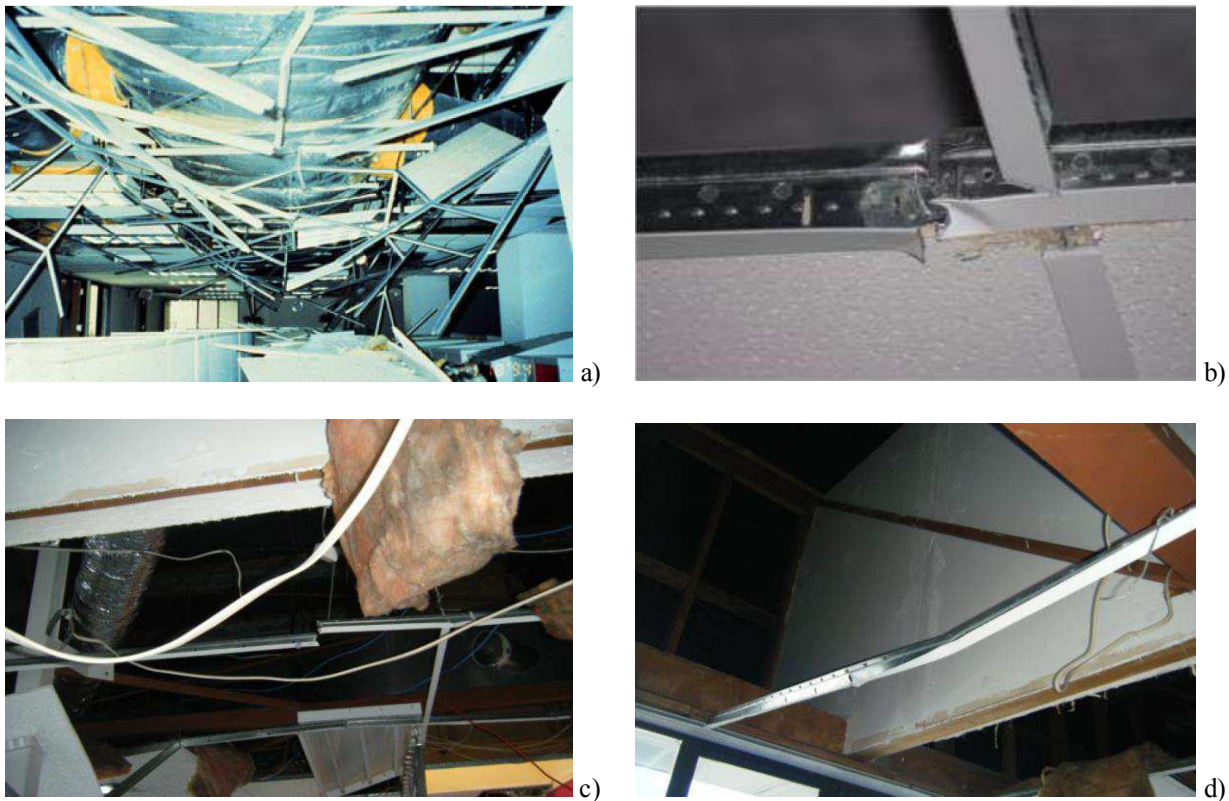


Fig. 9 a): Damage reported after the 1994 Northridge earthquake (Gilani et al., 2008); b): Damage reported after New Zealand earthquakes in 2010 and 2011. (Dhakal et al., 2013); c): Damage reported after the 22nd February 2011 Christchurch earthquake (Dhakal et al., 2011); d): Damage reported after the 22nd February 2011 Christchurch earthquake (Dhakal et al., 2011).

1.2.6 Interaction with other components

Suspended ceilings can be in contact with other elements situated at the ceiling level, such as electrical devices (e.g. light fixtures), terminals or services (e.g. diffusers, sprinkler heads). When earthquake occurs, these elements can interact and cause separations or poundings. This type of damage is very common when items are attached to the suspended ceiling and not independently supported, or when sufficient distances from the suspended ceiling are not respected. The damage due to interaction with lighting systems is reported in Fig. 10 b) and c). In Fig. 10 a), the interaction with suspended ceilings and bulkheads is indicated, while in Fig. 10 d) an electrical device falling down highlights the lack of proper support.



a)



b)



c)



d)

Fig. 10 a): Interaction between bulkheads and ceiling after New Zealand earthquakes in 2010-2011 (Dhakal et al., 2013); b) Damage reported after L'Aquila earthquake in 2009. (Magliulo, 2009); c) Ceiling panel dislodged in the Olive View Medical center, Sylmar, California, photo by Robert Reitherman. (Taghavi and Miranda, 2003); d) Damage reported after the Sichuan Earthquake, China, in 2008. (Glasgow et al., 2010)

1.3 Experimental campaigns

Experimental tests are the most direct way to evaluate the seismic performance of suspended ceilings. In particular, shake-table tests have been performed for more than 30 years.

In response to the increasing interest in seismic safety, a protocol for the seismic certification of non-structural components by shake-table testing was proposed by the International Code Council Evaluation Services (ICC-ES) Acceptance Criteria 156 (AC 156). The ICC-ES AC 156 protocol defines minimum requirements for the seismic certification of suspended ceilings by testing, as allowed by IBC (ICC 2006). The test is performed under a certain shaking intensity, that is

incrementally increased until reaching the collapse. Acceptance criteria for the certification are established, depending on structural, operational, and post-test assessment requirements. Testing reference spectra are based on the ASCE 7-10 (2010) equations to compute the seismic force on non-structural components and earthquake excitations are generated through spectra-matching procedures. The ceiling is qualified under the L_i level since it meets the established requirements during and after test, but it cannot encounter these same requirements at level L_{i+1} . The certification obtained allowed the use of the products in sites where the maximum seismic level expected is the one they are certified for. This testing methodology has become common among suspended ceilings manufacturers worldwide since it uses the only standardized protocol existing for non-structural components. Even if outside the United States there are no minimum requirements in terms of installation details or seismic performances, the AC 156 certification gives an industry standard answer to the request of seismic safety.

Since the protocol described above is not meant to give detailed information regarding the progression of damage during tests, researchers proposed the application of seismic fragility methodology to shake-table tests, maintaining the ICC-ES AC 156 protocol. According to this methodology, the most relevant damage states need to be identified and related to certain demand parameters (usually, the intensity of seismic input). Testing results are analysed to correlate these parameters to the damage observed. Finally, fragility curves can be plotted in function of one parameter, quantifying the probability of exceeding a certain damage state.

FEMA 2005 proposes a classification of progressive damage states for suspended ceilings that is presented in Table 1.

Table 1: Damage states according to FEMA 2005

Damage state	Description of damage			Repair
	Panel fall	Grid	Fixture damage	
Slight	Few	None	None	Put back panels
Moderate	Extensive	Disconnected or buckled at few locations	few	localized
Extensive	Extensive	Excessive buckling and partial collapse	Some fallen	Most or all panels
Complete	Extensive	Complete collapse and buckling	Many fallen	Complete replacement

Recently, static tests on full-scale specimens were also performed, using the same setup needed for shake-table tests. Finally, components of suspended ceilings were analysed in small scale, with the aim of evaluating the contribution of single elements to the overall behaviour.

In next section, a brief description of the main experimental tests regarding full-scale suspended ceilings or their components are presented. For each campaign the main findings and shortcomings are discussed.

1.3.1 Full-scale tests

In Table 2, experimental campaigns regarding full-scale specimens are listed for reference. In particular, the dimensions of the specimens tested and the location where the tests have been conducted are shown.

Table 2: Resume of the main experimental campaigns on full-scale suspended ceilings.

Reference	Location	Dimensions of the specimens
Rihal and Granneman (1984)	Facility of California Polytechnic State University	3.66x4.88 m
Yao (2000)	Laboratory at National Cheng Kung University, Tainan, Taiwan	1.2x4 m
Badillo (2003), Badillo et al. (2003), Badillo et al. (2004), Badillo et al. (2007)	University at Buffalo Structural Engineering and Earthquake Simulation Laboratory (SEESL) Buffalo	4.9 x 4.9 m
Gilani et al. (2008), Gilani et al. (2010)	University at Buffalo Structural Engineering and Earthquake Simulation Laboratory (SEESL) Buffalo	4.9x4.9 m
Reinhorn et al. (2010), Ryu et al. (2012)	University at Buffalo Structural Engineering and Earthquake Simulation Laboratory (SEESL)	6.1×16.3 m 6.1x6.1 m
Gilani and Takhirov (2011)	UC Berkeley-PEER	4.9x4.9 m
Gilani et al. (2012), Gilani et al. (2013), Gilani et al. (2014)	UC Berkeley-PEER	4.9x4.9 m
Magliulo et al. (2012), Magliulo et al. (2013a), Magliulo et al. (2013b), Pentangelo (2010)	Laboratory of Structural Engineering Department of University of Naples Federico II	3x3 m
Watakabe et al. (2012)	?	2.7x5.0 m
Soroushian et al. (2014), Soroushian et al. (2016c), Soroushian et al. (2016d)	University at Buffalo NEES E-Defense in Japan University of Nevada Reno NEES	6.1×16.3 m 6.1x6.1 m 10.1x12.2 m (16.2 m tall) 3x17.7 m 3x8.5 m (3.8 – 7.6 m height)
Gilani et al. (2015), Takhirov et al. (2015)	UC Berkeley-PEER	4.9x4.9 m
Pourali et al. (2015)	Structures laboratory at University of Canterbury	2.4x4.8 m
Gilani et al. (2017)	UC Berkeley-PEER	4.9x4.9 m
Pourali et al. (2017)	Structures laboratory at University of Canterbury	2.15x4.55 m
Masuzawa et al. (2017)	?	5.6x3.2 m

Rihal and Granneman (1984) performed the first dynamic tests on suspended ceilings, with harmonic excitations. The aim of the campaign was to evaluate the seismic behaviour of braced and unbraced suspended ceilings and their interaction with partition walls. Three different configurations were studied: isolated suspended ceilings and with partial- and full-height partition walls. The results outlined that a bracing realized with 45° splayed wires, connecting the runners to the soffit, was effective in stabilizing suspended ceilings with partial-height partition walls. The influence of compression post was also investigated, in fact specimens without vertical strut showed slackening of the splay wire bracing, damage at connection between main and cross tees and uplift of the ceiling. The compression post stiffened the system, consequently limited uplift was reported. In the specimens of suspended ceilings with full-height partition walls, details of the attachment with the wall were fundamental for the behaviour of the entire system. The findings of this campaign were used as starting point for the definition of prescriptive installation rules for suspended ceilings in seismic zones.

In Yao (2000), four types of suspended ceilings were studied with shake-table tests, in particular they differ for the presence of splay wire bracings and transverse support. Transverse supports are intended as short cross runners perpendicular to the longitudinal excitation direction. These elements were introduced to increase the lateral resistance of runners during the seismic motion. Results outlined that the natural frequency of suspended ceilings can be estimated with the simple pendulum formulation as long as the resonance period. Then, the presence of splay wires didn't affect the results. Moreover, the edge connectivity (meaning hanger wires in proximity to the edge of runners and pop rivets) can improve the seismic resistance. Finally, transverse supports are effective at reducing the lateral spread of runners and are proposed as solutions for wide suspended ceilings.

Tests described in Badillo (2003), Badillo et al. (2003), Badillo et al. (2004), Badillo et al. (2007) belong to the first extensive campaign performed with the AC 156 protocol. Twenty-seven shake-table tests were performed on six different configurations of specimens: suspended ceilings with normal sized and undersized panels and both with additional retainer clips, suspended ceilings with undersized panels and recycled grid components (components used in previous installations and disassembled) and finally suspended ceilings with normal sized panels without compression post. Results evidenced that undersized panels are the main cause of collapse of panels during the input, otherwise retainer clips can prevent tiles from popping out of the grid but direct the force to the main grid, possibly resulting in buckling. Moreover, recycled tees are an additional cause of damage to the grid. Finally, the results on the suspended ceilings without compression post can't confirm the improvement on the overall seismic behaviour. This campaign tested the configuration of suspended ceilings as proposed by current prescriptions and compared it with configurations adopted in the common practice. The overall results confirmed the better behaviour of suspended ceilings installed with Code prescriptions.

In Gilani et al. (2008) and Gilani et al. (2010), the AC 156 protocol was used to evaluate the performance of suspended ceilings with new perimeter details (see Fig. 4 c) in comparison to IBC (ICC 2006) standard perimeter installation. For qualification criteria, specimens with innovative details performed better than suspended ceilings installed with IBC prescriptions and were certified for being used in high seismicity zones. For the first time, the fragility approach was introduced in the testing methodology, but the damage affected only the panels. In fact, fragility curves were used to correlate the percentage area of fallen panels to the nominal spectral acceleration intensity. The fragility method is a powerful tool, in particular if combined with certification tests. However, repetitions of shake-table tests on the same specimen at increasing acceleration levels cause fatigue effects that can affect the results.

In Reinhorn et al. (2010) and Ryu et al. (2012), shake-tables were used to evaluate the seismic behaviour of 15 specimens and tests were performed according AC 156 protocol. 10 specimens are

6.1m large and 16.3m long, while the other specimens are square with dimension: 6.1m, 4.9m and 3.7m. The different dimensions of specimens permit to obtain an average behaviour of suspended ceilings, avoiding that size affects the results. Specimens differ for grid duty, panel weight and plenum height. Moreover, installation details as prescribed by IBC are also evaluated. Results outlined that the seismic force, applied in the longitudinal direction is transmitted along the main runners direction to the end of the tees or to the lateral bracing. Failures of pop-rivets are reported at the end of unrestrained tees. Moreover, the deflection of the main tee caused connection failure with cross tees. Additionally, the effect of lateral restraints was limited to the restrained runners and load could not be transferred from unrestrained tees to the lateral bracings because of their limited lateral stiffness. Finally, fragility curves helped to identify some critical aspects of suspended ceilings, such as multi-directional input motions, heavier panels, bigger ceiling areas and absence of lateral bracings.

In Gilani and Takhirov (2011), the innovative perimeter element described in Fig. 4 c) is tested to evaluate its contribution to the seismic behaviour of suspended ceilings. A similar work was developed in Gilani et al. (2008) and Gilani et al. (2010). Similar results were obtained, and specimens with the innovative perimeter element experienced lower accelerations and consequently less seismic demand on the grid. This work outlined the shortcomings of the AC 156 protocol, since some of the failures observed at the end of tests were not reported after past earthquakes. The authors recommended the development of a new test standard, that could take into account past experimental results and strong-motions recorded after past earthquakes. Moreover, they suggested to use performance levels in order to enhance the homogeneity of experimental results.

Important innovation in testing methodology was introduced by Gilani et al. (2012) through static loading tests for real-scale specimens, even if the purpose was limited to the study of the capacity of main runners. In Gilani et al. (2012), Gilani et al. (2013), Gilani et al. (2014) static and dynamic tests were performed on heavy- and intermediate-duty suspended ceiling systems. Static tests were performed with the same setup used for dynamic tests, which allows testing real-scale specimens and loading the main runner in his longitudinal direction. During shake-table tests, sensors were installed to measure strain of runners and displacements along the floating sides. Maximum tensile and compression capacity of runners obtained from static tests were compared to the strain recorded on runners during shake-table tests. The comparison outlined not only that during shake-table tests the maximum capacity of runners was not exceeded, but also that the displacement of runners didn't exceed the gap at perimeter prescribed by IBC. Moreover, results obtained confirmed the possibility to substitute intermediate- with heavy-duty main runners without affecting the seismic response of the suspended ceiling.

In Magliulo et al. (2012), Magliulo et al. (2013a), Magliulo et al. (2013b) and Pentangelo (2010), shake-table tests according to AC 156 protocol were performed to evaluate the behaviour of dry-wall continuous suspended ceilings with single- or double-frame. Specimens didn't show any damage at the end of the test and this result is coherent with the prevision of ASTM E580 that doesn't prescribe any requirements for this typology of suspended ceilings. In fact, plasterboards act as rigid elements and the high density of steel runners and vertical hangers increase the stiffness of the system. This work is really interesting in terms of confirming the rigidity of dry-wall suspended ceilings, even if the limited dimension of specimens can affect the results (see Table 2). Moreover, the use of AC 156 protocol gives the opportunity to compare the results with other experimental campaigns that used the same methodology. However, tests cannot be used as certification procedure outside USA, and it should be noticed that in the Italian context (where experiments were developed), the seismic force on non-structural components is computed with the European Code - Eurocode 8 (CEN 2013) formulation.

In Watakabe (2012), 8 specimens were tested, corresponding to a part of a collapsed dry-wall ceiling after the 2001 Geiyo Earthquake. Specimens differ for the position in the ceiling area, the hanging length and the flat or curved shape of the ceiling surface. Shake-table tests were performed using as input motion the numerical floor acceleration response obtained from the ground input motion of 2001 Geiyo Earthquake. As the main damage was caused by the behaviour of the clips that connected the primary to the secondary grid, an innovative clip was developed and tested. This innovative component improved the behaviour of the entire system, in fact damage was observed only during test with the previous clip.

In Soroushian et al. (2014), Soroushian et al. (2016c) and Soroushian et al. (2016d), the results obtained from different experimental campaigns were collected and compared. In the first campaign, performed at the University of Buffalo, fifteen specimens of suspended ceilings were tested, varying panel weight, ceiling area dimensions, presence of lateral restraints and uni- or multi-directional input motion. The second experimental campaign was developed in Japan, where two suspended ceilings were installed at the top floors of a 5-storeys building. The building was built on a shake-table and either fixed and different isolated base configurations were tested. In fact, the aim of the experimental campaign developed in Japan was to evaluate the influence of seismic base isolation and the contribution of seismic bracing to the global seismic behaviour of suspended ceilings. Bi- and tri-dimensional input motions were applied.

The last experimental campaign, performed at the University of Nevada Reno, tested 15 configurations of suspended ceilings located in a 2-storeys braced frame building spanning over 3 biaxial shake tables. Uni-directional input motion was applied to the shake tables. Specimens differ for boundary conditions, panel weight, sizes of ceiling area and presence of bracing system. The aim of the work was not only to identify the collapse mechanism of specimens but also to investigate the interaction with other non-structural components.

The results obtained from the three experimental campaigns permit to evaluate the failure mechanisms and outline the weakest elements of suspended ceilings. Since the installation details were different and also the dimensions of specimens and characteristics of the filtering structure, a good insight into the seismic behaviour of suspended ceilings can be given. The main damages observed regarded misalignment or falling of tiles, damage at perimeter, regarding both pop rivets or other perimeter connection elements, failure of grid joints, damage of hanger wires or lateral bracing and finally pounding of sprinkler heads at the ceiling tile. In particular, pop rivets failed at early stage of seismic motion. Moreover, in case of perimeter connection element, a sufficient edge length of perimeter trim should be provided. Then, the failure of grid connection results in partial or complete failure of the entire suspended ceiling system. Finally, in case of sprinkler heads, flexible drop pipes are suggested to be used to reduce pounding, as the gap dimensions suggested by American prescriptions don't avoid the local ruptures at the borders.

In Gilani et al. (2015) and Takhirov et al. (2015), the experimental approach presented in Gilani et al. (2012) was replicated. A real-scale suspended ceiling was subjected to monotonic load and the capacity of the main runners was investigated. Moreover, shake-table tests with AC 156 protocol were developed on three specimens of suspended ceiling. The first specimen is installed according to ASTM E580 prescriptions. The second and the third specimens used a shorter edge and differ for the installation of the innovative perimeter connection presented in Fig. 4 c) or the use of pop-rivets in the fixed sides. The results of shake-table tests proved that suspended ceilings with the innovative connection elements performed better than the suspended ceiling with prescribed installation. The results obtained in the static tests are used to evaluate the force acting on the main runners at the last stage of dynamic tests. It is demonstrated that the force on main runners is greater than the minimum capacity required by ASTM E580 and these conclusions highlighted the need to improve this prescriptive rule as it cannot avoid the damage of main runners during earthquake.

In Pourali et al. (2015), real-scale specimens were studied via shake-table tests, with the aim of evaluating the effects of different loading configurations and connections at perimeter. In particular, in two specimens main runners were placed in the direction of loading, while for other two specimens cross tees were loaded. Moreover, in the free edges, the perimeter connection elements presented in Fig. 4 c) were installed, except for one specimen that hasn't any connection element at the end of grid. Results outlined that the perimeter element tested reduced the displacement of the ceiling and the accumulation of forces in the grid. As the aim of the test was to investigate the load transmission through the grid and the amplification at different points, a sinusoidal input motion was applied. The work is a good starting point for an insight into the seismic assessment of suspended ceilings and it applies a valid approach for dynamic evaluation.

In Gilani et al. (2017), for the first time the performance of metal panel ceilings was compared to dry-wall suspended ceilings. Shake-table tests were performed according to the AC 156 protocol. Metal panel suspended ceilings were tested with both code-prescribed details at perimeter and with the element shown in Fig. 4 c). As already demonstrated in Gilani et al. (2008), Gilani et al. (2010), Gilani and Takhirov (2011), Gilani et al. (2015) and Takhirov et al. (2015), the innovative perimeter element limit the displacement of the suspended ceilings and their seismic performance is improved than with prescribed perimeter installations. The results of dry-wall ceilings confirmed the high stiffness of this typology of suspended ceilings, as already evaluated by Magliulo et al. (2012), Magliulo et al. (2013a), Magliulo et al. (2013b) and Pentangelo (2010) and considered in ASTM E580.

In Masuzawa et al. (2017), specimens of metal suspended ceilings with gypsum boards were tested via uni-directional shake-table tests. The configurations included the interaction with plasterboard walls and two metal elements are studied with the aim of reinforcing the ceilings. The first element is positioned between the panel and the main runner and has a T-shape. The second element has L-shape, it is installed at the perimeter and connects end panels to perimeter trims and plasterboard walls. The experimental results allow to compare an unrestrained specimen with specimens with U-element and T-element in addition. The performance of the reinforced specimens confirm that the innovative elements are effective at avoiding the fall of panels. This work provides innovative solutions to avoid the dislodging or fall of tiles and also the pounding with wall at perimeters. However, these elements are visible at the extrados of the tiles, implying a potential esthetical issue. In Pourali et al. (2017), fully-floating suspended ceilings were tested for the first time and unidirectional shake-table tests were performed. Specimens were realized with a 15 cm-gap at perimeter, packed with an elastic acoustic isolation material. Displacement at perimeter was measured during sinusoidal input motion, varying frequencies in relation to the fundamental frequency of the specimen. Input motion with frequency close to the resonant frequency caused pounding of the specimen against the surrounding structure but, in general, the foam placed in the gap improve the behaviour of the specimen, in particular regarding maximum displacements and accelerations. This work was developed for research purposes and in fact the protocol applied allowed to investigate the assessment of the specimen without looking for a certification level. The idea to realize a fully-floating suspended ceiling is innovative if considering its application in a seismic zone. Moreover, the study of a material or an element to be installed at the perimeter but not with constraining purposes can be the starting point of the definition of a new approach for the seismic protection of suspended ceilings.

In Yao et Chen (2017), twelve specimens were investigated through shake-table tests. The applied input motion is unidirectional, except for some specimens in which a vertical input component was also applied. The specimens differ for dimensions, configurations or main and cross runners and presence of lateral bracing and compression post. The input motions applied were derived from the AC 156 protocol. Results outlined that splay wires carry only a small portion of lateral inertial force and the situation is worst with vertical input; moreover the compression post is not effective in

limiting the vertical displacements. Damage was observed at the connection between runners, as one detached from the other. A solution was proposed and proved to be effective: it implies the restraining of the ends of the main runners with hanger wires, positioned in proximity to the joint. For the first time the efficiency of lateral bracing, as proposed by the American prescriptive rules, is critically evaluated. The experimental approach to evaluate the force acting on the bracing is effective and can be applied for other lateral bracing solutions.

1.3.2 Tests on components

The attention to the performance of ceiling components during earthquake starts with the identification of their local damage after seismic actions. Moreover, components can be tested in small-scale, requiring limited expense and instruments. The advantage in studying ceiling components is that eventual critical behaviours of components can be identified and modified independently of the entire system. Otherwise, it is not always easy to correlate the behaviour of a single component to the global seismic response and an additional real-scale test need to be performed. Furthermore, there isn't any protocol available for testing components.

The first evaluation of components was presented in Shephard and Shepphird (1990). Main and cross runners were tested with tension, compression and bending tests and pull-out tests were conducted on hanger wires. A test rig was built to develop this experimental campaign. The results of compression and tension tests outlined the weakness of the runners in correspondence to the slot, where the section area is reduced and resulted in buckling or tearing of the runner. For hanger wires tests, almost all the specimens showed tearing of the web of the section, except for few cases where the hook pulled out of its seating hole. This work gives only qualitative results, in terms of description of damage on specimens.

In Paganotti et al. (2011), it was indicated for the first time the lack of standards to test all the components in tension and compression. Consequently, researchers need to decide autonomously the configuration of specimens and the testing protocol, eventually affecting the final results. This experimental campaign regard runners, wires and inner and perimeter joints. Joints were reproduced connecting splices of runners and were placed on a vertical timber table and steel brackets were fixed on it to avoid displacement of runners during test and to address the load in the joint. In tension, the bayonet (defined as the part of the joint that is attached to runner) torn out from the runner to which was connected with rivets, while in compression, the connection was affected by buckling, meaning out-of-plane rotation of superior and inferior bayonets. Regarding the riveted connection of runners at perimeter, the tests caused the rupture of flange of the runner in the connection point with the rivet and consequently the detachment of the runner, possibly leading to fall of panels in real conditions. Soroushian et al. (2015a) presented an extensive experimental campaign regarding ceiling joints, hanger wires and panels. Ceiling joints were tested with shear and bending tests along major and minor axis of specimens. For each specimen both monotonic and cyclic tests were performed and the protocol applied to cyclic tests was designed especially for displacement sensitive non-structural components, indeed it was maintained for all cyclic tests on joints. Steel connection pieces and clamping plates were used in the rig to obtain the most realistic and accurate boundary conditions. Similarly to Paganotti et al. (2011), splices of runners were connected each other and to a small slice of runner placed horizontally, therefore load was applied vertically. Shear tests caused the complete tearing of web and flange of the slice of runner in one case, while in the opposite direction of the joint tests resulted in bending of the joint and partial detachment of the bayonet from the runners. Results outlined that in both configurations, the failure mechanisms reported could cause collapse of the ceiling. Results obtained from bending tests highlighted the small rotational strength in both loading directions. Moreover, at the final stage specimens reported localized rupture in one case and partial

bending of joints when load was applied in the other direction. Finally, results from the experimental campaign appeared not to be affected by the loading protocol or the typology of runner tested. Tensile tests were performed on hanger wires. All specimens reached the failure after having reduced their cross-sectional area, namely necking. Results were compared to the minimum tensile resistance indicated in ASTM E580 for hanger and splay wires, and specimens were found to comply the prescriptions. It was also observed that the results could vary depending on the attachment to the structure or to the supporting elements. Finally, the interaction of piping systems with tiles was simulated with monotonic tests. In the tile tested, drilled-in holes were realized to replicate the location of sprinkler system heads and a metal pipe was pushed through the tile. The tearing force was found to be independent of the displacement and need to be considered when studying the effect of piping pounding to the tiles.

In Soroushian et al. (2015c), monotonic tension and compression tests and cyclic tests were conducted to investigate the behaviour of connections between suspended ceilings and their perimeter. In particular, tests were conducted on three different configurations of connection: connection with pop-rivet and connection with the innovative perimeter element presented in Fig. 4 c with and without partition-attach screw. Moreover, main runners and cross runners were tested in the experimental configuration. The connector attached without screw had the smallest tension and compression capacity and failed after twisting around the perimeter trim. A similar rupture was obtained when a screw was installed, even if for a larger displacement. The installation of two screws can improve the behaviour of the connector at perimeter and postpone the failure that was experienced in the test. The pop-rivet connections, differently from Paganotti et al. (2011), failed for shearing off of pop rivets in compression, showing limited capacity if compared to the bearing capacity of the runners. Connections with pop rivets and perimeter connectors attached with a screw have similar tension capacity, but the pop rivet have smaller capacity in compression. Finally, less compression capacity was observed when cross runners were used in the tests, but the results are only dependent to the perimeter installation that changes according to the tee dimensions.

In Soroushian et al. (2016b), the axial capacity of suspended ceiling joints was tested with monotonic and cyclic tests. Specimens were obtained connecting slices of main and cross runners, similarly to Paganotti et al. (2011). The protocol for the cyclic tests was the same as the previous experimental campaign regarding suspended ceiling joints (Soroushian et al. 2015). In compression, failure was related to the buckling of the joint and only in few cases, tearing of the slot in the perpendicular member was observed. In tension, tearing of the hook located at the end of the plate was observed until it reached complete permanent bending. Specimens were divided in two groups depending on their axial capacity due to the joint condition. Finally, a numerical model was developed with the software OpenSees (Mazzoni et al. 2006). In detail, the joint was modelled combining a “zeroLength” element with the “Pinching4” uniaxial material, that permitted to simulate the hysteresis response as well as degradations for repetitive cyclic loadings.

Finally, in Dhakal and Pourali (2016) static tests were developed in cross tee connections (previously in Paganotti et al. 2011) other than main tee connections, from two different manufacturers. For both typologies, failure in tension was caused by tearing of the slot in the transversal runner, while in compression buckling of the joint caused compression and deformation of it. For main tee joints, the resistance in compression was higher than in tension, while for cross tee joints was the opposite, in fact the behaviour of cross tee connections in compression and main tee connections in tension was found to be critical. Dhakal and Pourali (2016) confirmed the lack of standards for testing of suspended ceiling joints, that are considered one the weakest parts of suspended ceiling systems. Moreover, while tensile tests indications are provided for runners, compressions evaluation is not mentioned.

1.3.3 Comments

From the experimental campaigns previously presented, some comments can be expressed regarding both methodology and results.

All tests on suspended ceilings are shake-table tests and most of them are part of a certification (also defined qualification) process with AC 156 protocol. This procedure has some shortcomings (Gilani and Takhirov, 2009), since some of the failures observed during tests were different from the actual earthquake damage. In fact, the rigidity of test frames need to be high enough to avoid amplification of the input in correspondence to the resonant period of the frame. Moreover, a specific input for suspended ceilings should be provided as the actual load is supposed to be applied to elements connected to the frame only in few points. Furthermore, specimens are usually tested different times at increasing shaking intensity, until collapse is reached. This procedure causes suspended ceilings to be over tested and results are not reliable as they are affected by iterated loading histories.

Gatscher and Caldwell (2017) observed that certification is costly prohibitive for non-structural elements whose components are frequently modified, as it needs to be repeated each time a variation is introduced. Finally, this certification process should not be allowed in countries different from the IBC (ICC 2006) pertinence areas. In fact, reference spectra used in AC 156 protocol are derived from the formulation proposed in ASCE 7-10 (2010) for computing the force on non-structural components. The protocol described in AC 156 can be used outside USA to test suspended ceilings, if the aim is having a common methodology that facilitates the comparison of results. Otherwise, a specific protocol needs to be based on local Codes formulations.

In relation to earthquake resistance evaluation, experimental tests can be divided in two categories, depending on their purpose: qualifying or fragility tests. Qualifying tests can be considered “pass or fail” tests, as their aim is not explicitly to give information on the mechanical properties of the specimens (Gilani and Takhirov, 2009) rather than prove that its performance comply to the Code prescriptions. Fragility tests, instead, are meant to give detailed information about the behaviour of the specimens, following the damage process and correlating damage mechanisms to mechanical parameters. In some tests, a fragility approach is used to analyse the results obtained from qualifying tests but fragility tests require a specific protocol to develop this procedure (for example, FEMA 461). In fact, the most relevant difficulty with shake-table tests is related to the choice of an appropriate input as their results are strictly dependent on it. Moreover, shake-table tests can be uni-, bi- or tri-axials and the history applied to the specimen depends on the combination chosen of the accelerograms in the different directions. Therefore, the same test can be difficultly reproduced elsewhere and the results obtained from shake-table tests where different inputs are applied are not consistent and cannot be compared. Finally, suspended ceilings should be tested under both vertical and horizontal motions (Bachman, 2008) since the stability of panels is mainly affected from the vertical component of seismic inputs. However, not all the shake-tables available for testing can apply 3D motions.

Most of the suspended ceilings studied are installed with ASTM E580 prescriptions, that are consequently been evaluated. The ASTM E580 seismic bracing was studied by Badillo et al. (2007) and the results didn't confirm clearly the contribution of the vertical strut to the global seismic resistance; moreover, the inefficiency of splay wires was evidenced in Yao et Chen (2017).

Many experimental campaigns proposed different perimeter configurations. Perimeter trim dimensions, connection elements and gap length need to be taken into account. In fact, the gap should be wide enough to accommodate the runners' displacement induced by the seismic motion. The innovative perimeter connectors proposed in Fig. 4 c) performed better than prescribed installation elements, as confirmed by many experimental tests (Gilani et al., 2008; Gilani and Takhirov, 2011; Soroushian et al., 2015a; Gilani et al., 2015; Gilani et al., 2017). These elements are installed in both

fixed and free perimeter sides and represent an alternative installation to screws and pop rivets connecting runners to perimeter trim, allowing also to reduce its edge length.

Continuous plasterboards suspended ceilings exhibited stiffer behaviour than tee-grid suspended ceilings (Magliulo et al., 2012; Gilani et al., 2017). In fact, usually seismic restraints are not applied and also ASTM E580 exempts this category from the prescribed details.

The most common strategy to improve the seismic behaviour of suspended ceiling is to apply one or more restraints connecting the suspended ceiling to the structure above and to the walls, so to increase its resistance and let it follow the movements of the main structure during the seismic motion. This technique causes the suspended ceiling to be more rigid and consequently concentrate more seismic load on it.

Opposite to this approach, in Robson et al. (2014) a fully-floating suspended ceiling was presented and studied. This typology of suspended ceilings can be observed in many projects worldwide, as it is frequently chosen for aesthetic reasons. From a seismic point of view, it allows the suspended ceiling to act as a pendulum, with the assumptions of negligible lateral stiffness and axial load carried by hanger wires. In this configuration, the forces induced by earthquake in the floor slab are not transferred to the suspended ceiling mass, that remains in its original position. With the aim of limiting the consequences of resonance phenomena, Pourali et al. (2017) introduced a viscoelastic foam layer at the boundary perimeters of a fully floating ceiling to prevent pounding. This innovative solution hasn't been applied yet to commercial products but can be an alternative and more achievable solution to the request of seismic resistance of suspended ceilings. Additionally, adequate distances between the suspension wires and equipment or piping systems should be indicated. In fact, in modern buildings with large dimensions (as hospital, airports, shopping centres etc.) the solution with lateral bracings can be an issue as many piping systems, HVAC equipment and conduits are attached to the soffit, resulting in lack of space for the installation of lateral bracings according their prescribed condition.

1.4 Numerical and analytical models of suspended ceilings

Only few numerical models of suspended ceilings have been presented in Literature and mostly combined with the results of experimental campaigns, that allow their calibration.

The first simplified model of suspended ceilings was described by Yao (2000) and it was developed with the commercial software Ansys (1993). The modelling was realized for suspended ceilings with and without the installation of the lateral bracing to evaluate the different behaviour of the ceiling. The model represents the ceilings tested with their limited dimensions (1.2x4 m). Cable elements are used to simulate the hanger wires, while runners are simulated with beam elements, in particular cross runners are connected with pin-ends to the main runners. The mass of tiles is considered but their stiffness is neglected. Finally, the perimeter trim is simulated with rollers. The fundamental frequency of the system without lateral bracing was found to be similar to a pendulum and the lateral bracing in the model caused higher fundamental frequency. The model is characterized by 5% of damping, a sine sweep excitation was applied and it outlined two realistic modes of vibration. The first mode represents a simultaneous displacement of the ceiling in its plane, while the other mode indicates a vertical deformation of runners and can be representative in case of large vertical motions. When considering the lateral bracing, the mode shapes are similar except for the higher natural frequencies. In Ryu et al. (2012) a simplified analytical unidirectional model is developed after the experimental campaign presented in paragraph 1.3.1. The suspended ceiling is represented in the longitudinal

direction as a multi-pendulum, where the main runner splices are modelled with slip-lock springs, moreover connection springs simulate the connection at perimeter and an external force is considered to model the restraint to the motion given by walls. The stiffness of the hanger wires is simulated assuming small displacements and considering a lumped mass from a tributary area and the assigned void height. The equation of motion is written for a multi-degree of freedom system whose dimension depends on the number of main runners considered. The equation is solved according to the condition applied to the experimental tests performed. The numerical results have a good correspondence to the experimental ones except for an interval where the large displacement of main runners causes failure of the cross runner pop-rivet, not considered in the model. In the transverse direction the ceiling is modelled as a supported beam with additional support springs representing the horizontal restrainers. In fact it takes into account the configuration of tiles and cross tees and their end and lateral restraints. The beam is simplified as a single degree of freedom element and the equation of the motion is written and solved with the experimental parameters in order to make a comparison of the results. The model cannot catch the contribution of a higher mode shown in the experimental results but can capture both magnitude and vibrations.

In Echevarria et al. (2012) a detailed model is realized with the software SAP2000 (2011). The T-grid is represented with beam elements with two different cross sections for main and cross runners, while for tiles a lumped mass is placed at the centre and connected rigidly to the four corners. The hanger wires and the lateral bracings are modelled with the Hook Link element, resisting only to tensile forces. Finally, the horizontal gap and the eventual uplift of tiles are considered with Friction Isolator Link. The elements used in the model represent accurately the typical installation as prescribed by ASTM E580. In the model the suspended ceiling is hung to a composite supporting floor deck in order to consider the effects in the model of the out-of-plane vibrations of the supporting floor. Two dimensions of ceilings are investigated: 13.4 m² and 72.8 m², and the contribution of the lateral bracing is evaluated. Non-linear time histories are performed with artificial acceleration records. The inputs are derived from the AC 156 spectrum regarding the horizontal direction while for the vertical direction the spectrum was produced according to the Chapter 23 of ASCE 7-10. Before performing the analysis the accelerations were subjected to filtering and baseline correction. Damage criteria are established according to the maximum displacement allowed in the horizontal direction (to get the closure of the gap between the tile and the runner) and vertical direction (uplift of the tile greater than the height of the runner). Finally, the numerical results are analysed developing fragility curves. The unseating of the runners at perimeter causes higher probability of damage in unbraced systems, but when only the tile uplift is considered, it is outlined that the damage is related to lower values of acceleration for the braced system. Moreover, the larger ceiling is less vulnerable to tile dislodgement but for the unbraced system it happens for lower acceleration than the smaller ceiling. The work highlights also some limitations of the model, mainly regarding the ability to capture other failures regarding the components, e.g. failure of inner connections or at perimeter.

Zaghi et al. (2016) presented a detailed numerical model of suspended ceilings realized with the software OpenSees, in fact this software was found to be a powerful tool to describe the progression of damage of the ceilings, not caught by the commercial programs previously cited. The characteristics of the elements in the model are chosen in order to calibrate it with the results obtained from the tests performed by Ryu et al. (2012) on a ceiling whose dimensions are 6.10x6.10 m. The tiles are modelled or with shell elements or with an x-shape assembly with a centre joint semirigidly connected to the corner with contact elements. A contact element was specially created to reproduce the gap between the tiles and the runners, the friction related to the displacement of the runners, the impact of the tiles to the runners and the energy dissipation of this phenomenon. A beam element is used to model the runners, in detail a zero length element is represented by two nodes placed in the same position connected by a model that represents the uniaxial behaviour of the joints. The hanger

wires are described with beam elements and a PDelta effect that reduces the stiffness in order to take into account their initial slack. The splay wires of the lateral bracing are modelled with truss elements with tension-only elastic material. The same element created to model the contact between the tiles and the runners is used to represent the boundary conditions in both horizontal and vertical directions. An elastic material with progressive damage was used to model the damage of fixed connections at perimeter. The first limit state considered to study the ceiling is the damage of tiles: the dislodgment is defined by the uplift of at least one corner of the tile combined to the closure of its horizontal gap. The axial deformation and the gap closure of the element used to model the tiles were monitored in real-time for each step of the analysis in order to remove the element if the criteria were encountered. Another limit state considered is the damage at the grid connection, that is defined as the exceed in the capacity rotation of the joints and causes the remove of tee beam and tile involved. Then wire damage was considered for both hanger wires and lateral bracing wires. Also the damage due to unseating at the boundary is taken into account monitoring the deformation of the tee beam, when they exceed the seat width of the perimeter trim, the end tee beam and the adjacent tiles are removed. Finally, the damage of the rigid connection at perimeter is considered. The numerical model have a good correspondence with the experimental results but couldn't describe the propagation of the damage. In fact, the damage changed the mass distribution and the damage propagation increased the uncertainties in the behaviour of the components, reducing the accuracy. Moreover, it was outlined that the use of different accelerograms results in different damage.

The analytical model proposed by Ryu et al. (2012) and the numerical model by Yao (2000) are simple enough to be potentially applied in the design practice, even if they cannot describe accurately the variety of damage that can be reported by the ceiling in consequence to the seismic motion. On the other side, the most accurate numerical models proposed by Echevarria et al. (2012) and Zaghi et al. (2016) have been realized for research purpose. In fact, the accuracy reached by Zaghi et al. (2016) was obtained with the use of a non-commercial finite element program and the realization of particular elements that accurately describe the behaviour of the ceiling components. The main limitation in the realization of the numerical models of suspended ceilings is that they need to be calibrated on the results of experimental campaign. Moreover, they cannot easily be adapted to ceilings with different configurations from the one considered, as the overall behaviour is complex and it is affected by the interaction of the different components. When realizing a numerical model for suspended ceilings, a balance between simplification and accuracy needs to be find according to the aim of its development.

1.5 Analytical and numerical analysis methods

The response of suspended ceilings installed at different levels of a building can be studied with the same analysis methods proposed for non-structural components in general. They are divided in two main categories: direct analysis and cascading analysis methods.

Direct analyses include the complete models of suspended ceilings and structure, at which ground motions are applied. This approach is the most exhaustive, but it requires very complex numerical modelling, with many elements and large number of degrees of freedom. Moreover, the properties of suspended ceilings and structural elements are different in terms of order of magnitudes, potentially leading to ill-conditioned models. Then, it should be noticed that the properties of suspended ceilings are often not known (due to lack of experimental data). Complexity of models is also increased by the high number of attachment points of suspended ceilings to the structure, from which the seismic

motion is transferred. Finally, this method can be applied only at final stages of the building design, when both structural and non-structural components are defined. Furthermore, it should be considered that the engineers who design the structure are not the same persons in charge of the non-structural project. Direct numerical analyses are usually developed for research purpose and their efficiency is enhanced calibrating the model properties on experimental results. Indeed, their complexity makes them useless for design support in manufacturer companies.

Cascading analyses are defined as the uncoupled analyses of structure and suspended ceilings within a building. This method implies the study of structure and suspended ceilings separately and can be applied only when structure and components have great difference of mass. First, ground motions are applied to the main structure, without considering the non-structural component attached to it rather than its mass, if significant. Then, the response of the structure at the level where the suspended ceiling is installed is applied as input motion to it. When the response of the main structure is intended as Floor Response Spectra, the method is defined FRS method. The dynamic analyses can be carried out considering natural ground motions, recorded after past earthquakes or artificial seismic shocks, generated by a commercial software in order to meet the spectrum compatibility requirement with the design elastic spectrum, defined by Codes. Natural ground motions are useful in a research context as they can reproduce a real seismic event on the model, but the results are affected by the peculiar dynamic characteristics of the input motion. The second approach is the most accurate for design purposes, as the elastic design spectrum is defined in all main Seismic Codes. This method can be potentially used by designers as it simplifies the computational cost required by a complete modelling, without significantly affecting the accuracy of the results.

Simplified formulations are also proposed in literature to generate directly the floor response spectra from the acceleration spectrum at the base of the building. The methodologies proposed aim to be implemented in Codes expressions to increase their accuracy. The formulation proposed by Calvi and Sullivan (2014) considers the dynamic filtering of the structure and the resonance phenomena due to natural period of vibrations of the non-structural component close to the period of the vibration of the structure. Moreover, the expression takes into account the elastic damping of the non-structural component and the influence of the earthquake intensity as it pushes the structure in a non-linear range. The methodology has been improved in Welch and Sullivan (2017) to better compute the nonlinear response in multi-degrees of freedom systems, even if the complexity of the approach limits its applicability in Codes indications. A simplified method based on dynamic principles is proposed also by Vukobratovic and Fajfar (2016). In their methodology the inelastic behaviour of the structure is related to the fundamental mode and is taken into account with a reduction factor applied to the ductility value, otherwise the other modes are considered elastic. Finally, a pushover-based procedure is used to define the parameters related to the inelastic structure. Lucchini et al. (2016) present a probabilistic seismic demand model for non structural components that includes an equation to generate floor response spectra based on properties for the peak value of the spectra calculated in the period regions that correspond to the vibration periods of the structure. The amplitude of the peaks is affected by the period and the damping of the non-structural component, other than the floor level and earthquake intensity level. In Degli Abbatì et al. (2017) the formulation proposed starts with the definition of two significant points: the peak floor acceleration point, that represents the spectral ordinate at period $T=0$ and the maximum spectral acceleration for the considered period of vibration of the structure. An analytical formulation connects these two points to build the spectra, and amplification and correction factors are introduced to consider the peak of the acceleration response spectra in correspondence of the natural vibration period and the damping reduction to the elastic spectrum. This study considers the structure elastic but indicates a formulation for equivalent elastic behaviour that takes into account the elongation of the period of vibration due to the non-linear behaviour and the variations of damping.

The simplified methods previously described were developed as instruments to study the behaviour of the non-structural components installed in a building, overcoming the computational cost required by classic dynamic analysis, meaning both cascading methods or the even more complex direct methods. The simplified formulations for computing the seismic response spectra are also meant to be integrated in the Code expressions, that will be presented in the next paragraph.

1.6 Evaluation of seismic forces on suspended ceilings according to Codes

In the main Codes worldwide there is a section dedicated to the computation of the seismic force acting on non-structural components. The value obtained is usually used to design the anchorage of the non-structural component to the building, as the purpose of the Code is maintaining life safety and the common practice is to limit the displacement of the component. The assumption is that the non-structural component and the building are uncoupled and the seismic demand on the non-structural component is evaluated through the definition of an equivalent static design force applied to the centre of mass of the component.

In this section, the formulations for calculating seismic forces acting on non-structural components are presented, according to the European Code (EC8), the American Code (ASCE 7-10) and the New Zealand Code (NZ 1170.5:2004). The formulation proposed in the European Code computes an equivalent horizontal force applied to the centre of gravity of the element, while in the American and New Zealand Code a vertical component of the equivalent seismic force is also considered.

European Code - Eurocode 8 (CEN 2013)

$F_a = \frac{\alpha S}{\left(\frac{q_a}{\gamma_a}\right)} \left[3 \frac{\left(1 + \frac{z}{H}\right)}{\left(1 + \left(1 - \frac{T_a}{T_1}\right)^2\right)} - 0.5 \right] W_p$	α	Ratio of the design ground acceleration on type A ground, to the acceleration of ground g
	S	Soil factor
	T_a	Fundamental vibration period of the non-structural component
	T_1	Fundamental vibration period of the building in the relevant direction
	z	Height of the non-structural component above the level of application of the seismic action
	H	Building height measured from the foundation or from the top of a rigid basement
	W_p	Weight of the component
	γ_a	Importance factor of the component
	q_a	Behaviour factor of the component

American Code – ASCE 7-10

$F_{ph} = \frac{0.4a_p S_{DS}}{\left(\frac{R_p}{I_p}\right)} \left(1 + 2\frac{z}{h}\right) W_p$ $F_{pv} = \pm 0.2 S_{DS} W_p$	a_p	Component amplification factor
	S_{DS}	Design earthquake spectral response acceleration at short period
	R_p	Component response modification factor
	I_p	Component importance factor
	z	Height of the structure at point of attachment of non-structural component
	h	Average roof height of structure relative to the base elevation
	W_p	Operating weight of non-structural component

New Zealand Code – (NZ 1170.5:2004)

$F_{ph} = C(0) C_{Hi} C_i(T_p) C_{ph} R_p W_p$ $F_{pv} = C_{pv} C_{vd} R_p W_p$	$C(0)$	The site hazard coefficient for $T=0$
	C_{Hi}	Floor height coefficient for Level i
	T_p	Period of the component
	C_{ph}	Horizontal design coefficient of the component
	C_{pv}	Component vertical response factor
	C_{vd}	Vertical design action coefficient
	R_p	Component risk factor
	W_p	Weight of the component

For all the formulations here presented the force acting on the component is proportional to the weight of the component and function of the height of the floor where the component is attached within the building, moreover a reduction factor is introduced to considers the nonlinear response of the component. The Codes don't analyse specifically every typology of non-structural component, rather than considering its importance with a specific factor related to the risk induced by the failure of the component. From a dynamic point of view, non-structural components can be categorized in displacement- or acceleration-sensitive and it is clear that the same force-based approach proposed by Codes cannot be used for both of them, without representing a shortcoming. Another disadvantage of the indications of the Codes is that designers don't have any specifications on the most vulnerable subcomponents in non-structural elements that need to be verified. This is particularly important in suspended ceilings as only the bracing systems are designed to resist to the design seismic force, even if other weakest components are reported from past earthquakes or Literature.

Finally, the reliability of seismic codes in regards to non-structural components was investigated by Sullivan et al. (2013). In this work, non-linear time history analyses were developed on two cantilever RC wall buildings, with 8 and 20 storeys. Accelerograms were scaled to be spectrum compatible with the EC8 type 1 spectrum for soil type C and $PGA=0.2$ g for the 8-storey structure while $PGA=0.4$ g was chosen for the 20-storey structure. Numerical floor response spectra at the roof level, with 2% and 5% damping of non-structural components, were compared to seismic codes results. The comparison outlined that, particularly on short-periods, the acceleration demand on non-structural components with Codes approach was greatly underestimated.

1.7 Conclusions

The aim of this chapter was to describe and comment the main research activities and practice about suspended ceilings, with focus on their seismic behaviour.

Reports from earthquake damages are useful to define the weakest components or inadequate installation techniques. The outcomes can be used to develop specific experimental campaigns in order to verify the performance of the weakest components or to compare the damage resulting from full-scale testing to real damages.

Experimental tests are a powerful tool to verify the performance of suspended ceilings with traditional installation technique or to investigate the improvement given by new components to the overall behaviour. However, many limitations of the actual procedure were reported, being evident that the widespread certification process couldn't be applied with research purposes without being modified. The most important observations regarding the experimental methodology used so far can be summarized in key points:

- Sufficient number of specimens should be tested for each input, in order to avoid the influence of previous loading history on the results;
- Scale effect can be avoided using adequate dimensions of specimens;
- The testing methodology should be changed in order to obtain results independent from the input chosen;
- Tests should be more economic and easily replicable.

The considerations previously presented addressed this work to the study for an innovative testing methodology able to overcome the limitations of the actual procedure. This approach is meant to characterize the behaviour of the suspended ceilings rather than certify them for a seismic level. The characterization is obtained providing seismic performance characteristics, i.e. strength degradation, damping, energy dissipation, ductility. Moreover, the damage configurations and weakest components can also be outlined. The idea behind this innovative approach is to provide engineers with experimental data representative of the capacity of the suspended ceilings that can be used in the design stage.

The numerical models of suspended ceilings described in this chapter have different levels of complexity and have been all developed for research purposes. Their accuracy in fact is strictly related to the calibration performed on results of previous experimental campaigns. For this reason, they cannot be easily adapted to other configurations or installation techniques and their purpose is restricted to research and innovation context. On the other side, the FRS method represents a powerful instrument to calculate the seismic force on suspended ceilings installed at different levels of a building, as it simplifies the computational cost required by complete modelling, maintaining good accuracy in the results. Moreover, in recent Literature, some works deal with further improvement about the computation of FRS. These procedures are intended to be integrated in the Seismic Codes, in order to overcome limitations of actual formulations.

2 Experimental and numerical characterization of suspended ceiling joints behaviour

The results of experimental tests and the damages observed after earthquakes worldwide show that the failure of ceiling joints is a common cause of collapse of the entire suspended ceiling system. As outlined by previous authors, the characterization of suspended ceiling joints is crucial for a detailed study of suspended ceilings. While many experimental campaigns have been developed with the aim of studying perimeter connections only few experimental and numerical data are available about inner joints.

The work here presented aims to characterize the behaviour of two typologies of suspended ceiling joints, whose shape is different from all the suspended ceiling joints already investigated in literature. In detail, a particular type of joints designed to resist to seismic load (named ‘seismic’) is compared to ‘standard’ joints. The experimental campaign has also the aim of evaluating the influence of the runners’ section on the behaviour of the joints and for this reason many specimens for each configuration were tested. Finally, the behaviour of main runner joints was also investigated. For each category at least 3 monotonic tension and 3 cyclic tests were performed. Cyclic tests were executed to get an insight on the behaviour of joints subjected to repetitive and alternate loading input and a specific loading protocol was developed and applied. Results permit to investigate the resistance and dissipative behaviour of ‘standard’ or ‘seismic’ joints. Moreover, a numerical model using the software OpenSees (Mazzoni et al. 2006) was developed for both standard and seismic joints and calibrated on the base of experimental results.

The results allow to achieve a better understanding of inner joints behaviour in the perspective of improving the global seismic behaviour of suspended ceilings.

2.1 Description of the specimens

To connect main runners to cross tees or cross tees to cross tees, two typologies of joints were studied. The first typology is named ‘*standard*’ and is applied to different runners. In this work, the categories of runners considered are named “Base24” and “Base15”. The second type of joints included in the study is defined ‘*seismic*’ and it is only applied to “Base24” products. Finally, the main runner joints belongs to the “Base24” category.

In the category Base24 the cross sections of main and cross runners have similar shape but different height: main runner is 38 mm high while cross runner is 32 mm high (Fig. 14 a) and b), their base is 24 mm wide. Moreover, the main runner is made from 3.5/10 steel sheet, while for cross runners the thickness of the sheet is 3/10.

As evidenced in Fig. 12, the standard joint has a large end fold in which a notch is positioned (external notch). Conversely, the seismic joint (Fig. 13) has an end Z fold and in the centre of the plate a notch is positioned (internal notch). The differences between standard and seismic joints are the shape of the end fold and the position of the notches. In fact, the interlocking capacity is demanded to the end Z fold for seismic joints and to the internal notch for standard

joints. In seismic joints, the internal notch gives an additional contribution to the interlocking resistance.

Base24 main runners have typical length of 3800 mm. For longer spans they are connected each other with typical joints, illustrated in Fig. 14: at the end of the runner a tab is provided and it is realized with a hook and a dimple positioned at the end.

In Base15 the main and cross runners have the same shape, illustrated in Fig. 15. The section of the runners is 41.5 mm high and 14.5 mm wide. The particular shape of the bottom of the runner allows to conceal the grid once tiles are installed. As previously indicated, Base15 is provided only with standard joints, whose shape is identical to the standard joints of Base24. In Fig. 16 detailed pictures of Base15 runners and joints are provided.



Fig. 11: Base24 cross section: a) main runner b) cross tee.

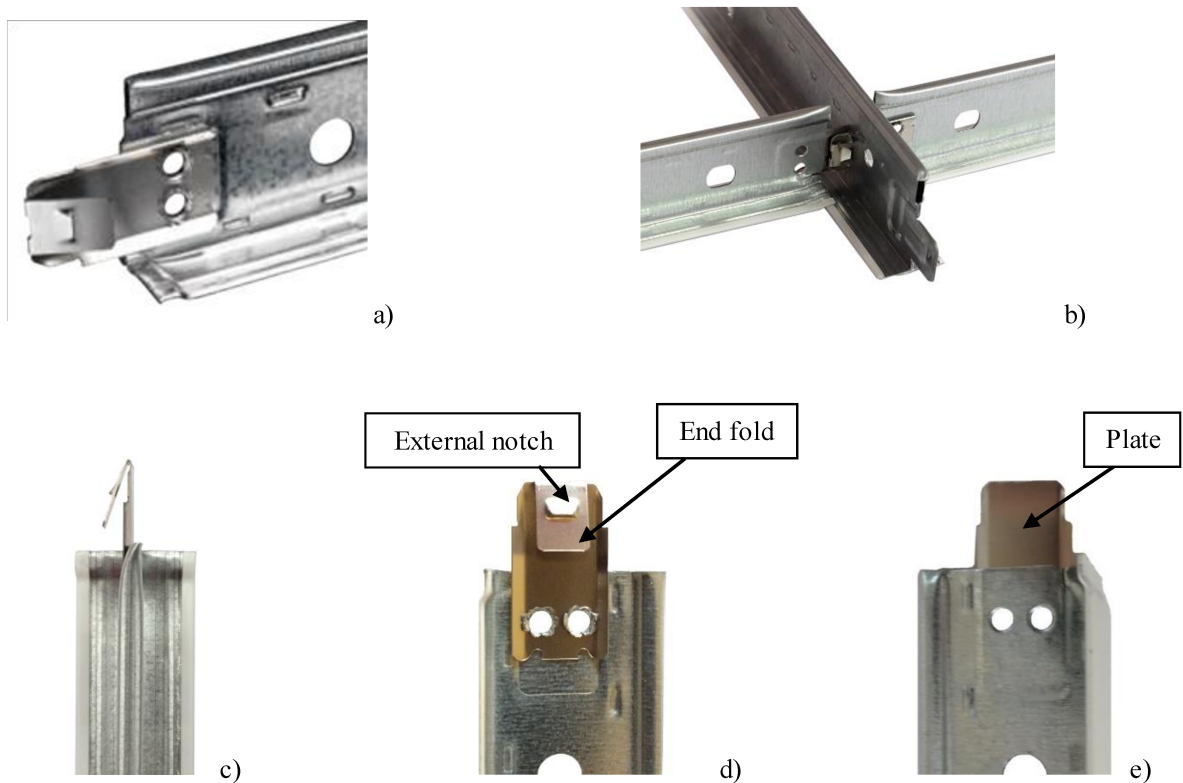


Fig. 12: Base24 standard joint: a) axonometric view, b) overview, c) top view, d) left side view, e) right side view.

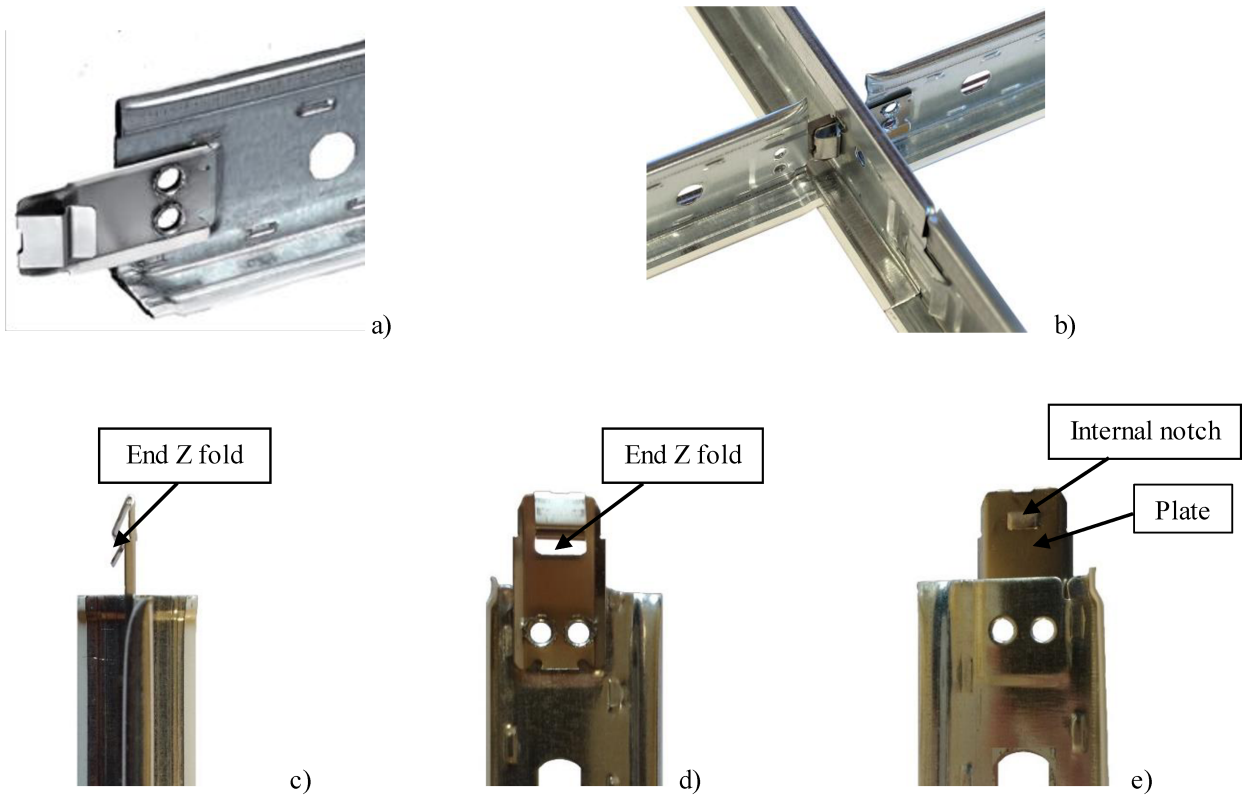


Fig. 13: Base24 seismic joint: a) axonometric view, b) overview, c) top view, d) left side view, e) right side view.

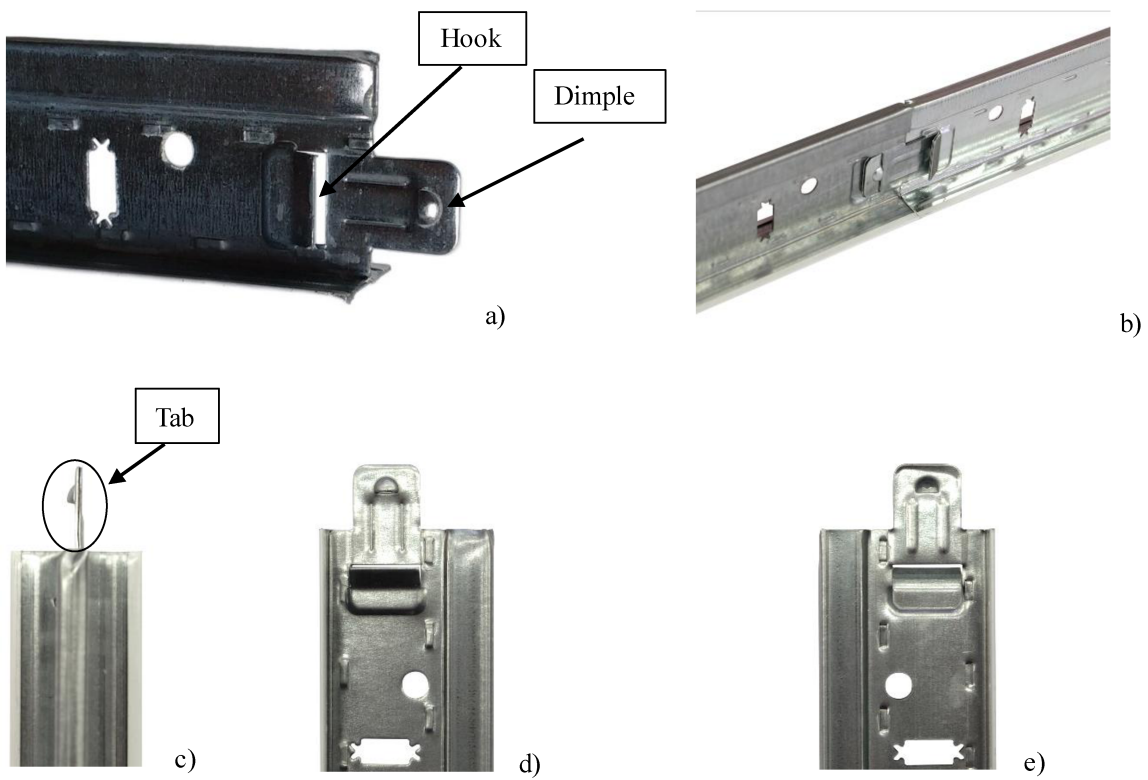


Fig. 14: Main runner joint: a) axonometric view, b) overview, c) top view, d) left side view, e) right side view.

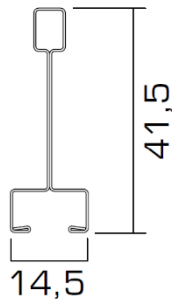


Fig. 15: Base15 cross section.

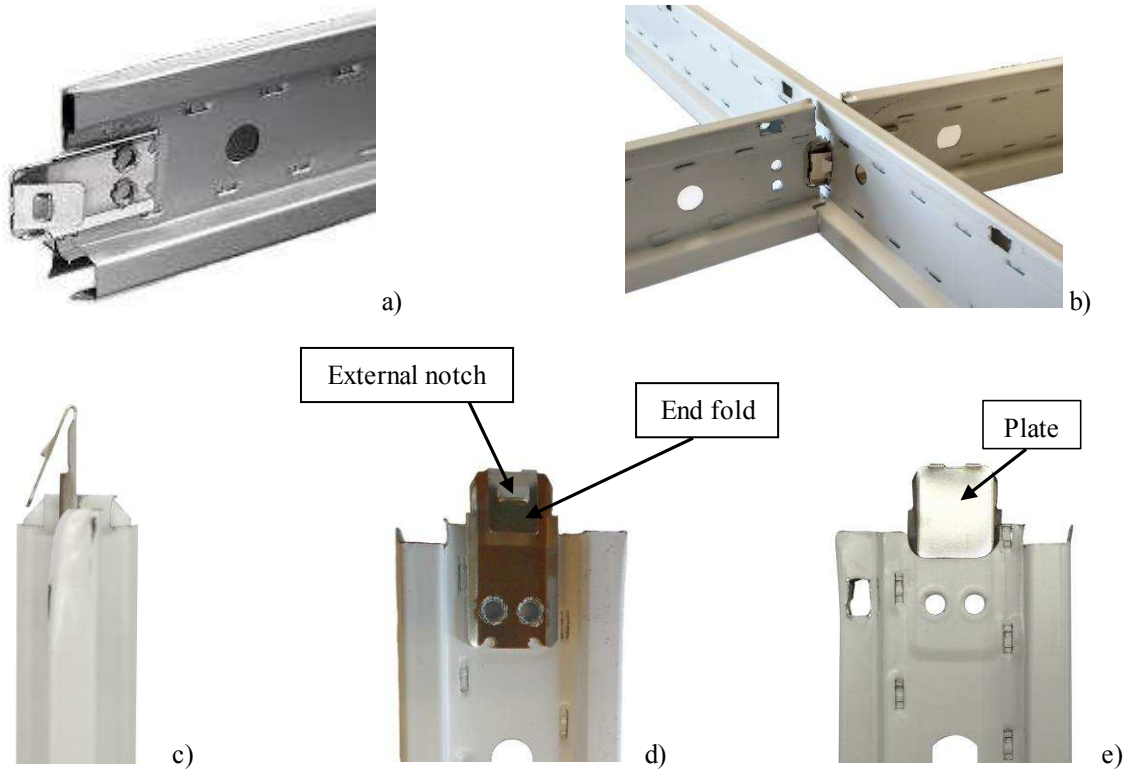


Fig. 16: Base15 standard joint: a) axonometric view, b) overview, c) top view, d) left side view, e) right side view.

Experimental tests include the evaluation of both standard and seismic joints installed in different categories of runners, moreover main runner joints are investigated. In Table 3, a summary of the tested specimens is presented. In detail: *Runner type* is the name of the runners; axial load is applied to *Longitudinal runners*, while *Transversal runners* are included only to represent real conditions; *Joint type* indicates standard or seismic joints and *Label* is the abbreviation used to identify the specimens.

Table 3: Summary of specimens tested.

Runner type	Longitudinal runner	Transversal runner	Joint type	Label
Base15	Main runner (=Cross runner)	Main runner (=Cross runner)	Standard	M-M Stand B15
Base24	Cross runner	Main runner	Standard	C-M Stand B24
	Cross runner	Cross runner	Standard	C-C Stand B24
	Cross runner	Main runner	Seismic	C-M Seism B24
	Cross runner	Cross runner	Seismic	C-C Seism B24
	Main runner		Main	M Main B24

2.2 Experimental tests

2.2.1 Test setup and procedure

Tests were performed with the Universal Machine Galdabini Sun 2500 (see Fig. 17), able to apply both tension and compression loadings. The load cell of the machine has a maximum load capacity equal to 3430 N. The lower vise keeps one end of the longitudinal runner blocked while the upper vise impose a vertical displacement to the other end. Elongation of specimens is measured directly by the strain gauges installed in the machine.

The transversal tee was held in place by clamps fixed to the machine support with a magnet. Thus, rotation of transversal runners and horizontal displacement of specimens were prevented and only the upper end of longitudinal tee was allowed to move reciprocally. Dimensions of specimens were chosen to better represent the real conditions. In fact, for M-M Stand B15 a length of 20 cm for transversal runners and 30 cm for longitudinal runners were selected. 30 cm-length runners were also chosen for M Main B24. For C-M Stand B24, C-C Stand B24, C-M Seism B24 and C-C Stand B24 20 cm-length transversal runners and 10 cm-length longitudinal runners were chosen.

As indicated in Table 4, for each typology of specimens at least 3 tension monotonic and 3 cyclic tests were conducted in displacement control. Specimens are identified with the final letter *m* if tested through monotonic tension test or *c* in case of cyclic test. In fact, monotonic tests have the aim to evaluate the collapse mechanism and estimate the yielding value of the joint. In relation to cyclic tests, the protocol chosen was adapted from the EN12512 (CEN 2006) protocol for cyclic quasi-static testing of connections in timber structures (see Fig. 18). The amplitude of cycles are function of the yielding value, previously evaluated trough monotonic tension tests. In detail, after two adjustment cycles, three cycles for each displacements are performed. The velocity of application of the displacement was chosen equal to 5 mm/min.



Fig. 17: Universal Machine Galdabini Sun 2500.

Table 4: Scheme of the experimental campaign

Name	Loading Protocol	Nr. of specimens
M-M Stand B15	Monotonic (m)	3
	Cyclic (c)	3
C-M Stand B24	Monotonic (m)	3
	Cyclic (c)	3
C-C Stand B24	Monotonic (m)	5
	Cyclic (c)	3
C-M Seism B24	Monotonic (m)	3
	Cyclic (c)	3
C-C Seism B24	Monotonic (m)	4
	Cyclic (c)	5
M Main B24	Monotonic (m)	8

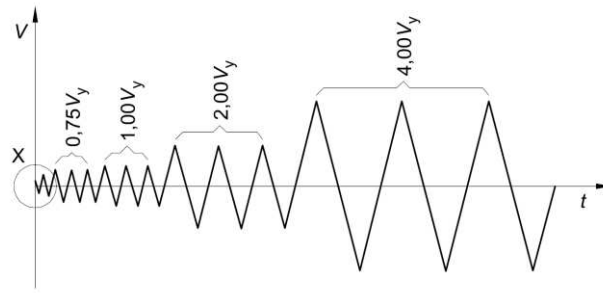


Fig. 18: EN12512 protocol (CEN 2006)

2.2.2 Test results

2.2.2.1 Base15 joints

Results obtained from tests on M-M Stand B15 joints are presented in Table 5, in Fig. 19 a) and b) force-displacement graphs are shown. As it can be noticed, tensile monotonic tests envelop quite well the corresponding cyclic tests, the lower mean value of cyclic tests could be explained with the progressive degradation of specimens. The specimens showed elasto-brittle behavior, with limited plastic deformation. The behaviour in compression is due to increasing of compressive strength when the gap between elements is closed. In cyclic tests, all the 4.5 mm-amplitude cycles could be executed, except for one test. In fact, while in compression there is no evident failure and only contact between runners is shown, the test is interrupted when in tension failure is encountered, in detail all specimens reached ~ 6 mm of final displacement. As it can be seen in Fig. 20, failure corresponds to tearing of the slot in the transversal tee.

Table 5: Experimental results on Base15 joints.

Monotonic tension test			Cyclic test				
Specimen	Force max [N]	Mean [N] Std. Dev. [%]	Specimen	Force max [N]	Mean [N] Std. Dev. [%]	Force min [N]	Mean [N] Std. Dev. [%]
M-M Stand B15_4m	397.14	365.06 9.7 %	M-M Stand B15_2c	339.73	357.99 11.8 %	-968.83	-995.42 20.6 %
M-M Stand B15_5m	371.23		M-M Stand B15_3c	406.24		-804.54	
M-M Stand B15_6m	326.81		M-M Stand B15_6c	328.00		-1212.87	

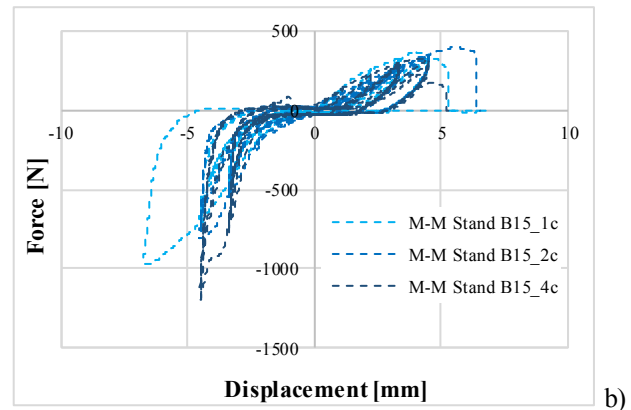
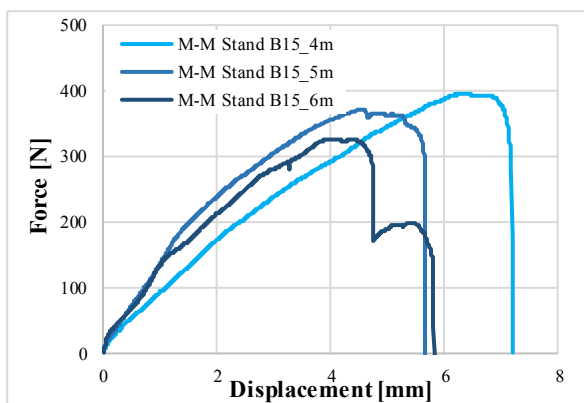


Fig. 19: Force-displacement response of M-M Stand B15 specimens subjected to a) monotonic and b) to cyclic test.



Fig. 20: Detail of damage on M-M Stand B15 specimen: tearing of the slot in the transversal runner.

2.2.2.2 Base24 Standard joint

Results obtained from monotonic tension and cyclic tests on standard joints in Base24 runners are presented in Table 6. In Fig. 21 a) and b) the force-displacement curves are indicated for C-M Stand B24 joints, while in Fig. 21 c) and d) the results for C-C Stand joints are illustrated. First, it can be noticed that the behaviours of C-M and C-C Stand B24 joints are similar, as well similar to the experimental results of M-M Stand B15. Otherwise, C-M Stand B24 joints have higher maximum resistance than C-C Stand B24 joints, both in compression and tension. In fact main runner's web is wider and thicker than cross runner's web, consequently the resistance to tearing of the slot is higher. All specimens were able to complete the first cycle in tension at 6.75 mm, but only one specimen reached -6.75 mm in compression.

In Fig. 22 details of joint's final configuration are illustrated. For both C-M and C-C Stand B24 joints, collapse was obtained after tearing of the slot in the transversal runner, in fact a similar failure was obtained from M-M Stand B15 joints. The resistance of C-M and C-C Stand B24 joints is inferior than M-M Stand B15 joints because of the shape of the runners and their mutual connection at the joint. In fact, as it can be seen comparing Fig. 11 b) to Fig. 16 b), Stand B24 joints allow a small gap at the connection with transversal runners and thus larger displacements are allowed during tests.

Table 6: Experimental results on Base24 runners with standard joints.

Monotonic tension test			Cyclic test				
Specimen	Force max [N]	Mean [N] <i>Std. Dev.</i> [%]	Specimen	Force max [N]	Mean [N] <i>Std. Dev.</i> [%]	Force min [N]	Mean [N] <i>Std. Dev.</i> [%]
C-M Stand B24_8m	446.69	377.20 20.0 %	C-M Stand B24_1c	301.24	364.80 18.7 %	-599.53	-599.40 19.8 %
C-M Stand B24_9m	388.22		C-M Stand B24_2c	356.49		-717.73	
C-M Stand B24_16m	296.70		C-M Stand B24_3c	436.66		-480.95	
C-C Stand B24_2m	304.90	328.29 10.2 %	C-C Stand B24_1c	361.78	340.07 6.5 %	-471.05	-530.80 9.8 %
C-C Stand B24_3m	362.60		C-C Stand B24_2c	340.74		-562.89	
C-C Stand B24_4m	318.80		C-C Stand B24_3c	317.68		-558.46	
C-C Stand B24_18m	291.02						
C-C Stand B24_19m	364.14						

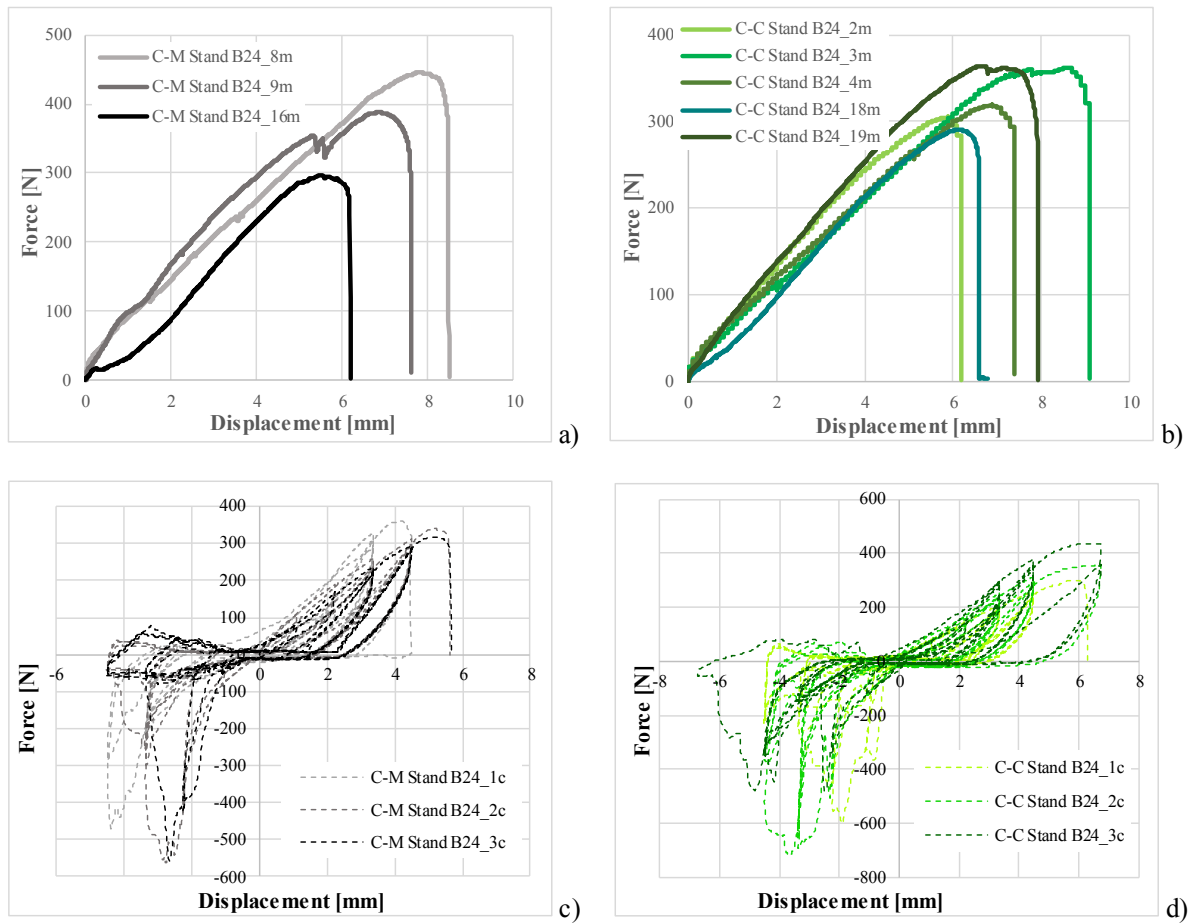


Fig. 21: Force-displacement response of: a) C-M Stand B24 joints and b) C-C Stand B24 joints subjected to monotonic test; c) C-M Stand B24 joints and d) C-C Stand B24 joints subjected to cyclic test.

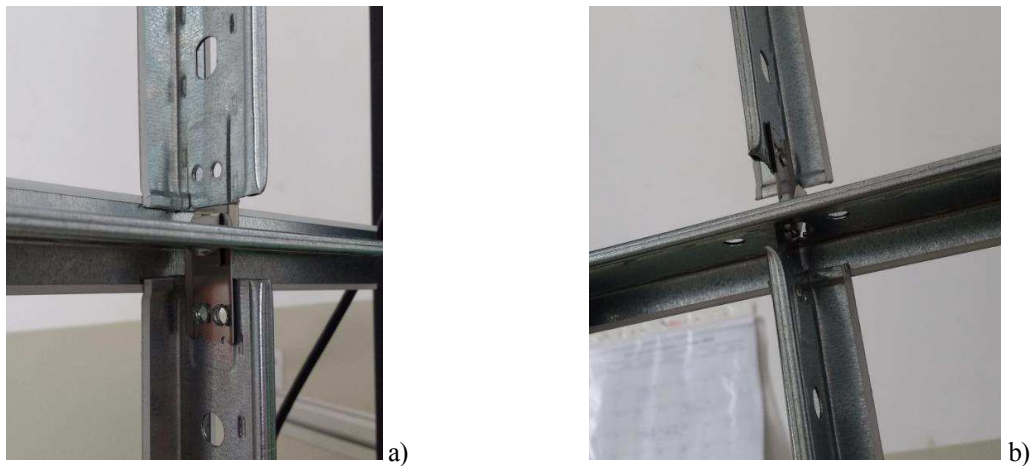


Fig. 22: Detail of the damaged configuration: tearing of the slot in the transversal runners, a) on the left side of the joint, during compression, b) on the right side of the joint, during tension.

2.2.2.3 Base24 Seismic joint

The results of monotonic tension and cyclic tests are indicated in Table 7, while in Fig. 23, the graphs representing force-displacement results are presented. C-M and C-C Seism B24 joints

have similar behaviour, regarding both maximum resistance in tension and compression. Only during cyclic tests C-M Seism B24 joints have higher resistance in tension.

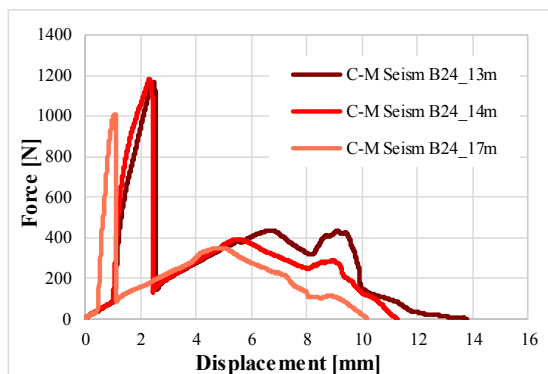
As it can be noticed in Fig. 23, the peak of resistance in tension corresponds to approximately 1200 N and the relative displacement is 2 mm for monotonic tension tests and 1 mm for cyclic tests. There are three changes of stiffness in tension: the first and the third parts have the same slope, that corresponds to the contribution of the end Z fold, while the peak of resistance is due to the internal notch that catches the other joint. The peak of resistance is clear only in the first cycles as the internal notch is not effective after being deformed.

The behavior in compression can be observed in Fig. 23 c) and d). In fact, it can be observed a peak of resistance at first cycles but the stiffness decreases rapidly at following cycles, due to tearing of the slot in the transversal tee. Moreover, the particular shape of the joint allows to maintain a residual resistance in tension after the first cycles. In details, after 2-3 mm of displacement in compression, the end Z fold starts to unhook and after the change of direction, it can't catch the transversal runner again and the end Z fold is deformed (see Fig. 24 a). The joint unhooks completely when -4.5 mm of displacement is reached.

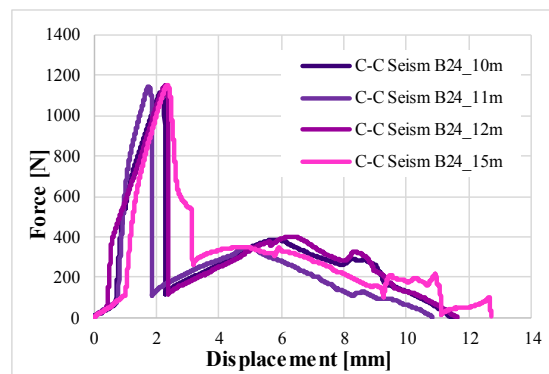
In Fig. 24 some details of damage during cyclic tests are shown, in fact the damage is similar for C-M Seism B24 and C-C Seism B24 specimens. For both typologies, failure is reached in tension when tearing of the slot is encountered, this stage corresponds also to complete deformation of both end Z fold and internal notch.

Table 7: Experimental results on Base24 runners with seismic joints.

Monotonic tension test			Cyclic test						
Specimen	Force max [N]	Mean [N] Std. Dev. [%]	Specimen	Force max [N]	Mean [N] Std. Dev. [%]	Force min [N]	Mean [N] Std. Dev. [%]		
C-M Seism B24_13m	1166.48	1119.91 8.4 %	C-M Seism B24_1c	1138.57	1343.35 13.2 %	-480.83	-438.09 10.8 %		
C-M Seism B24_14m	1181.67		C-M Seism B24_2c	1434.957		-387.49			
C-M Seism B24_17m	1011.57		C-M Seism B24_3c	1456.54		-445.95			
C-C Seism B24_10m	1118.90	1142.71 1.4 %	C-C Seism B24_1c	1187.11	1172.19 3.5 %	-492.99	-485.22 14.3 %		
C-C Seism B24_11m	1146.67		C-C Seism B24_2c	1203.61		-550.57			
C-C Seism B24_12m	1150.08		C-C Seism B24_3c	1125.85		-412.11			
C-C Seism B24_15m	1155.18								



a)



b)

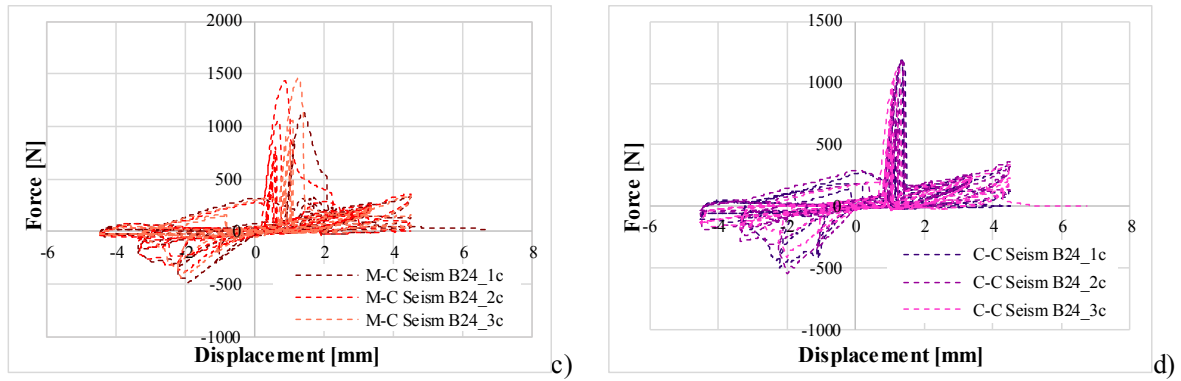


Fig. 23: Force-displacement response of: a) C-M Seism B24 joints and b) C-C Seism B24 joints subjected to monotonic test; c) C-M Seism B24 joints and d) C-C Seism B24 joints subjected to cyclic test.

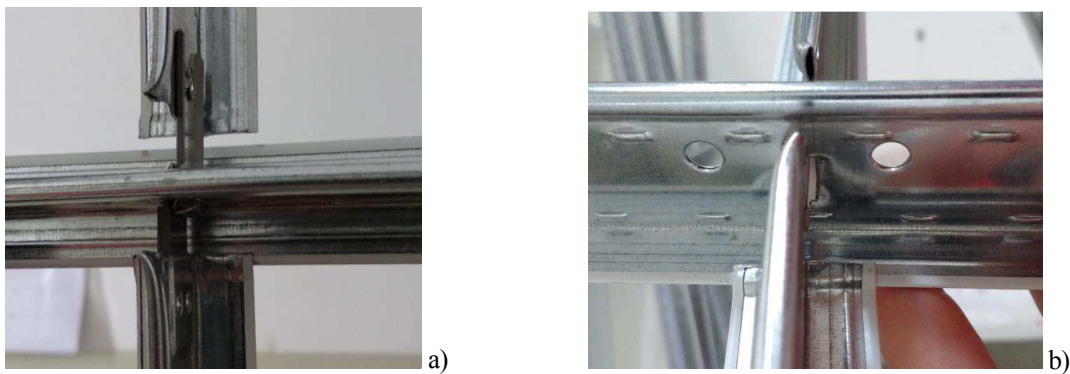


Fig. 24: Details of the damaged configuration of Seism B24 joints: a) deformation of the end Z fold due to repetition of cycles; b) tearing of the slot in the transversal runner.

2.2.2.4 Base24 Main Runner joint

In Table 8, the results obtained from monotonic tension tests on main runner joints are indicated. As it can be seen in Fig. 25, there is high variability in the results, although they have the same elasto-brittle behaviour. In detail, the peak of resistance corresponds to 1-2 mm of displacement.

In Fig. 26, the collapse mechanism of joints connecting main B24 runners is illustrated. In particular, collapse is related to complete tearing of the dimple.

Table 8: Experimental results on Base24 main runners joints.

Monotonic tension test		
Specimen	Force max [N]	Mean [N] Std. Dev. [%]
M Main B24_1	524.895	954.568 36.4 %
M Main B24_2	994.805	
M Main B24_3	1341.077	
M Main B24_4	1540.525	
M Main B24_5	854.035	
M Main B24_6	575.943	
M Main B24_7	954.59	
M Main B24_8	850.675	

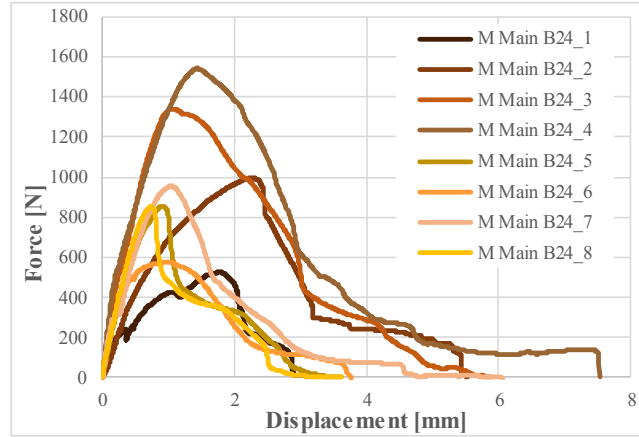


Fig. 25: Force-displacement response of M Main B24 joints specimens.



Fig. 26: Detail of damage of the dimple.

2.3 Numerical models of the joints

Experimental results were used to calibrate an hysteresis numerical model with the OpenSees software. Numerical models were subjected to the same experimental loading history using a displacement-controlled reversed cyclic analysis procedure. The calibration process of all typologies of joints was performed using the most significant experimental cycles.

The calibration of *Standard* joints was performed using a numerical model consisting of one truss element with Pinching4 material model (available in the software) to simulate the nonlinear behaviour of joints.

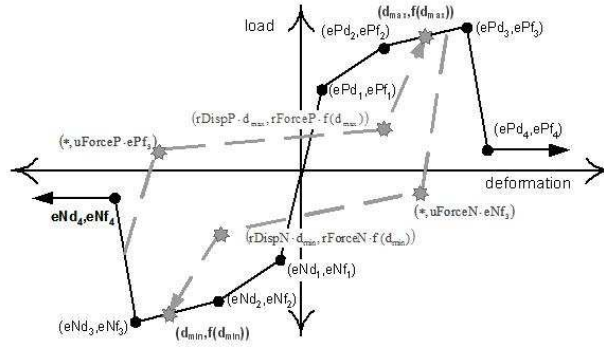


Fig. 27: Pinching4 material model OpenSees (Mazzoni et al. 2006)

Sixteen parameters (Fig. 27) are needed to describe the whole cyclic behaviour in tension and compression domain, accounting for hardening and softening and progressive decaying of mechanical properties with repetition of cycles. In particular positive and negative envelopes were defined by (Mazzoni et al. 2006):

- ePd_i : point values defining deformation points on the positive response envelope;
- ePf_i : point values defining force points on the positive response envelope;
- eNd_i : point values defining deformation points on the positive response envelope;
- eNf_i : point values defining force points on the negative response envelope.

Pinching cycles were defined by six parameters:

- $rDispP$: the ratio of the deformation at which reloading occurs to the maximum historic deformation demand;
- $rDispN$: the ratio of the deformation at which reloading occurs to the minimum historic deformation demand;
- $fForceP$: the ratio of the force at which reloading begins to force corresponding to the maximum historic deformation demand;
- $fForceN$: the ratio between the force at which reloading begins and the force corresponding to the minimum historic deformation demand;
- $uForceP$: the ratio of strength developed upon unloading from negative load to the maximum strength developed under monotonic loading;
- $uForceN$: the ratio of strength developed upon unloading from negative load to the minimum strength developed under monotonic loading.

Strength and stiffness degradation were not considered.

To better represent the experimental behaviour of *Seismic Base24* joints, a different numerical model with two springs in parallel was realized.

The peak of resistance observed in tension during seismic joints' tests was simulated using a truss with the hysteretic material model shown in Fig. 28.

The backbone curve needs for twelve parameters (Mazzoni et al. 2006):

- sip : stress (or force) at i-points ($i=1,2,3$) of the envelope in the positive direction;
- eip : strain (or deformation) at i-points ($i=1,2,3$) of the envelope in the positive direction;
- sin : stress (or force) at i-points ($i=1,2,3$) of the envelope in the negative direction;
- ein : strain (or deformation) at i-points ($i=1,2,3$) of the envelope in the negative direction;

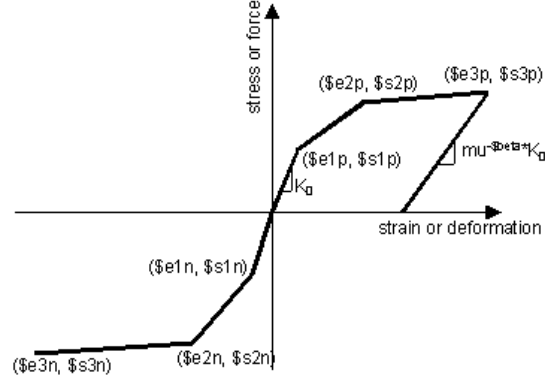


Fig. 28: Hysteretic material model OpenSees (Mazzoni et al. 2006)

The second spring was characterized by the pinching4 material model previously defined.

2.3.1 Base15

The numerical model for Standard Base15 joints was calibrated by fitting the corresponding experimental loading history. The near-collapse state reached at 4.5 mm defined the maximum cycle amplitude for the calibration. The parameters selected for the pinching4 material model are listed in Table 9.

The comparison of numerical and experimental cycles (Fig. 29 a) shows the good accuracy of the numerical model. In particular, the energy comparison graph (Fig. 29 b) shows that the discrepancy at near collapse condition is less than 5%.

Table 9: Pinching4 parameters for Standard Base15 joints

ePf_1	ePf_2	ePf_3	ePf_4	eNf_1	eNf_2	eNf_3	eNf_4	$rDispP$	$rForceP$	$uForceP$
0.01	0.27	0.35	0.38	-0.015	-0.09	-0.45	-0.70	0.56	0.12	-0.03
ePd_1	ePd_2	ePd_3	ePd_4	eNd_1	eNd_2	eNd_3	eNd_4	$rDispN$	$rForceN$	$uForceN$
0.03	2.24	3.35	4.50	-0.02	-1.70	-3.35	-4.50	0.66	0.14	-0.06

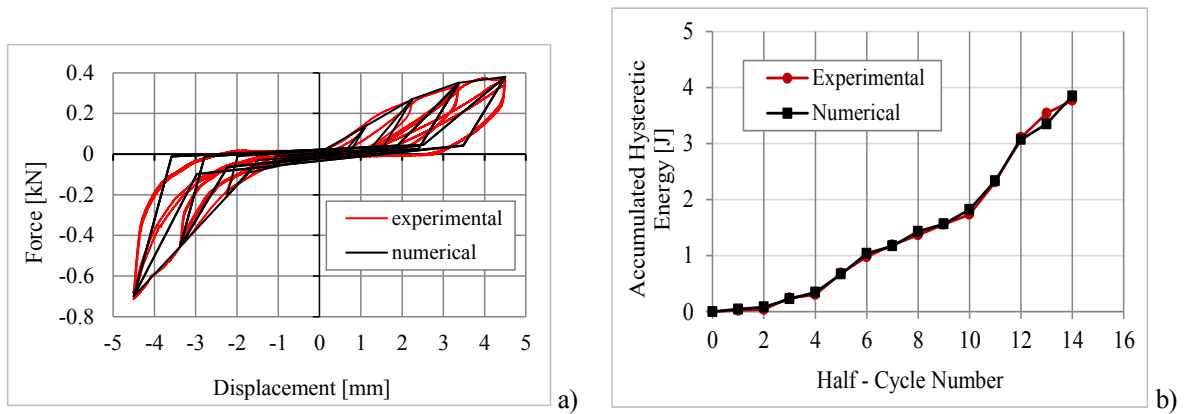


Fig. 29: Numerical calibration B15 joints: a) Hysteresis cycles comparison; b) Accumulated hysteretic energy comparison.

2.3.2 Base24 Standard

Tests on Base24 Standard joints were performed matching primary and secondary runners as listed in Table 10. As before, the numerical truss element representing the Standard Base24 joints was calibrated with reference to mean experimental loading history. The maximum amplitude cycle value is 3.375 mm for both specimens.

Table 10: Specimens used for Standard joint's tests

Commercial Line	Primary runner	Secondary runner	Joint type	Name
Base24	Main runner	Cross runner	Standard	C-M Stand B24
	Cross runner	Cross runner	Standard	C-C Stand B24

2.3.2.1 C-M Standard B24

The parameters used for the pinching4 material model for C-M Standard joints are listed in Table 11.

Also in this case, the hysteresis cycles comparison graph (Fig. 30 a) shows a good correspondence between numerical and experimental results. Energy graph (Fig. 30 b) points out a local overestimation of numerical values only in the first cycles (30%) and an average difference between numerical and experimental results less than 10%.

Table 11: Pinching4 parameters for C-M Standard B24 joints

ePf_1	ePf_2	ePf_3	ePf_4	eNf_1	eNf_2	eNf_3	eNf_4	$rDispP$	$rForceP$	$uForceP$
0.08	0.11	0.20	0.33	-0.02	-0.03	-0.08	-0.22	0.44	0.21	0.05
ePd_1	ePd_2	ePd_3	ePd_4	eNd_1	eNd_2	eNd_3	eNd_4	$rDispN$	$rForceN$	$uForceN$
0.29	0.90	2.00	3.38	-0.05	-0.90	-2.00	-3.38	0.59	0.18	-0.05

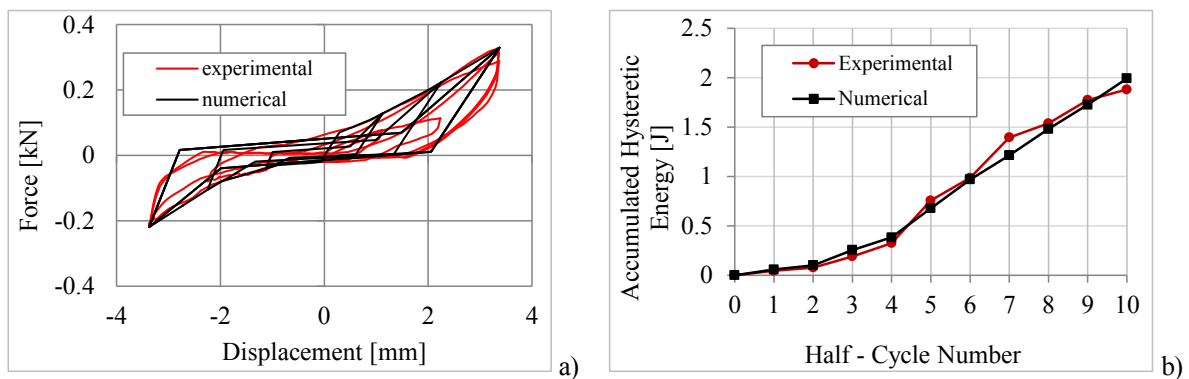


Fig. 30: Numerical calibration C-M Standard B24 joint: a) Hysteresis cycles comparison; b) Accumulated hysteretic energy comparison.

2.3.2.2 C-C Standard B24

The calibrated parameters used for the pinching4 material model for C-C Standard joints are listed in Table 12.

Fig. 31 a) and b) shows, respectively, the hysteresis cycle comparison and the accumulated hysteretic energy comparison.

Results reveal a good correspondence between numerical and experimental values, in terms of strength, stiffness and energy dissipation. In particular, the energy graph shows that numerical results never over-estimate the experimental ones and that the average deviation between them is less than 5%.

Table 12: Pinching4 parameters for C-C Standard B24 joints

ePf_1	ePf_2	ePf_3	ePf_4	eNf_1	eNf_2	eNf_3	eNf_4	$rDispP$	$rForceP$	$uForceP$
0.05	0.17	0.25	0.01	-0.05	-0.17	-0.60	-0.70	0.36	0.18	-0.13
ePd_1	ePd_2	ePd_3	ePd_4	eNd_1	eNd_2	eNd_3	eNd_4	$rDispN$	$rForceN$	$uForceN$
0.20	2.20	3.40	3.43	-0.04	-1.55	-2.82	-3.43	0.69	0.18	-0.02

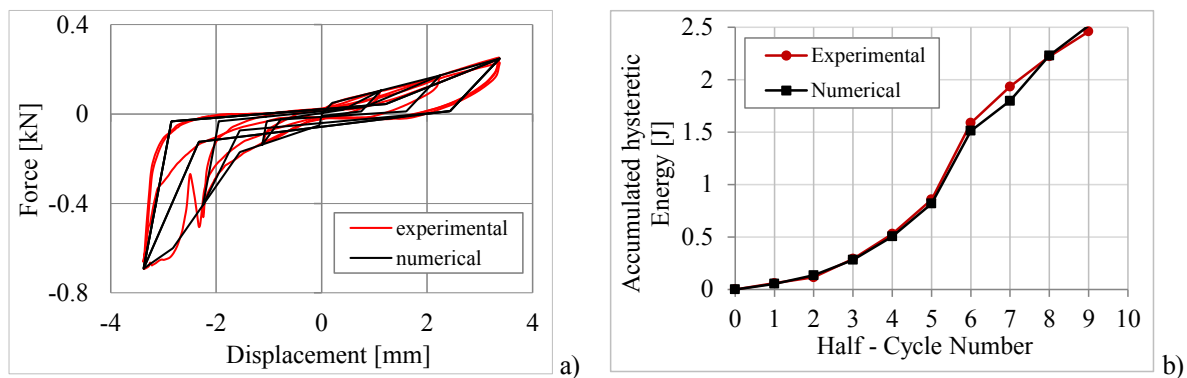


Fig. 31: Numerical calibration C-C Standard B24 joint: a) Hysteresis cycles comparison; b) Accumulated hysteretic energy comparison.

2.3.3 Base24 Seismic

Tests on Base24 Seismic joints were performed matching primary and secondary runners as listed in Table 13.

Their numerical model adopts two springs in parallel characterized respectively by pinching4 and hysteretic material model.

The behaviour of seismic joints is more complex than standard joints behaviour. In fact, after reaching the peak of resistance in tension and tearing of the slot in compression, secondary mechanisms were shown. Thus, the calibration of seismic joints was conducted considering only the cycles characterized by first-order resistance mechanism.

Table 13: Specimens used for Seismic joint's tests

Commercial Line	Primary runner	Secondary runner	Joint type	Name
Base24	Main runner	Cross runner	Seismic	C-M Seism B24
	Cross runner	Cross runner	Seismic	C-C Seism B24

2.3.3.1 C-M Seismic B24

The parameters used for the pinching4 and hysteretic material models for C-M Seismic joints are listed in Table 14 and Table 15.

Results shows a good correspondence between numerical and experimental values (Fig. 32 a) and b). Energy graph indicates a deviation value of local results equal to 20% and an average discrepancy between values of less than 10%.

Table 14: Pinching4 parameters for C-M Seismic B24 joints

ePf_1	ePf_2	ePf_3	ePf_4	eNf_1	eNf_2	eNf_3	eNf_4	$rDispP$	$rForceP$	$uForceP$
0.05	0.11	0.19	0.25	-0.03	-0.07	-0.44	-0.01	0.33	0.32	-0.05
ePd_1	ePd_2	ePd_3	ePd_4	eNd_1	eNd_2	eNd_3	eNd_4	$rDispN$	$rForceN$	$uForceN$
0.27	1.10	2.25	3.40	-0.05	-0.40	-2.22	-2.23	0.36	0.18	-0.18

Table 15: Hysteretic material parameters for C-M Seismic B24 joints

$e1P$	$e2P$	$e3P$	$e1N$	$e2N$	$e3N$
0.70	1.10	1.20	-0.01	-0.20	-0.35
$s1P$	$s2P$	$s3P$	$s1N$	$s2N$	$s3N$
0.01	0.90	0.025	-0.005	-0.006	-0.007

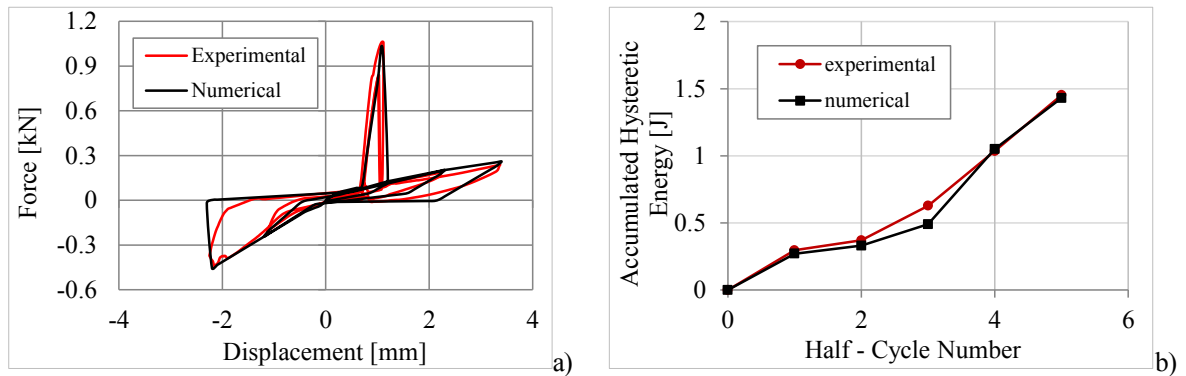


Fig. 32: Numerical calibration M-C Seism B24 joints: a) Hysteresis cycles comparison; b) Accumulated hysteretic energy comparison.

2.3.3.2 C-C Seismic B24

The parameters used for pinching4 and hysteretic material models for C-C Seismic joints are listed in Table 16 and Table 17.

The numerical model is able to represent the behavior of the seismic joint and, in particular, to capture the main mechanisms of resistance. The underestimation of the energy is due to the calibration of the resistance peak. However, the average deviation is less than 25%.

Table 16: Pinching4 parameters for C-C Seismic B24 joints

ePf_1	ePf_2	ePf_3	ePf_4	eNf_1	eNf_2	eNf_3	eNf_4	$rDispP$	$rForceP$	$uForceP$
0.05	0.10	0.19	0.25	-0.04	-0.08	-0.40	-0.01	0.45	0.35	0.05
ePd_1	ePd_2	ePd_3	ePd_4	eNd_1	eNd_2	eNd_3	eNd_4	$rDispN$	$rForceN$	$uForceN$
0.20	1.10	2.25	3.40	-0.06	-0.40	-2.10	-2.23	0.27	0.13	-0.05

Table 17: Hysteretic material parameters for C-C Seismic B24 joints

$e1P$	$e2P$	$e3P$	$e1N$	$e2N$	$e3N$
0.90	1.16	1.20	-0.05	-0.06	-0.15
$s1P$	$s2P$	$s3P$	$s1N$	$s2N$	$s3N$
0.01	1.10	0.01	-0.005	-0.006	-0.007

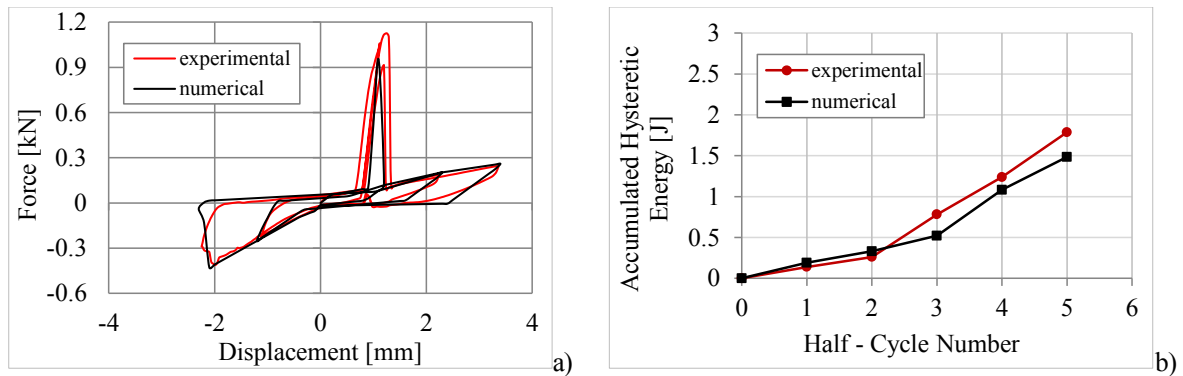


Fig. 33: Numerical calibration C-C Seism B24 joints: a) Hysteresis cycles comparison; b) Accumulated hysteretic energy comparison.

2.4 Conclusions

Experimental tests were performed on standard and seismic joints, other than joints connecting main runners. Monotonic tension tests were conducted to evaluate the yielding point and the collapse mechanism in tension. Then, cyclic quasi static tests were carried out to investigate the global behaviour of joints subjected to alternate loadings. The aim of the experimental campaign was to evaluate and compare the experimental behaviour of standard and seismic joints. Moreover, joints were installed in different typologies of runners, in order to investigate the factors affecting the behaviour of joints.

All joints had elastic-brittle behaviour and reached high values of strength. This behaviour was particularly evident in seismic joints, that showed three-four times the maximum resistance in tension than standard joints. In fact, the seismic joint is affected by the contribution of the internal notch and, when the notch reaches failure, the joints become equivalent to standard joints, both in load and ultimate displacement capacity.

The behaviour in compression is critical, in particular regarding Base24 specimens that experienced great degradation of resistance after the first cycles.

Joints between main runners in Base24 specimens showed great variability of results. The reason can be related to an improper locking of the runners or to the mechanics of the joints.

In relation to the seismic joints, it can be noticed that the design approach aims to react to the earthquake motion with higher resistance rather than a more dissipative behaviour. However, the high resistance is maintained only for a limited displacement and the resultant behaviour is elastic-brittle. A new design should be provided to the seismic joints in order to achieve a better resistance in compression and a global dissipative behaviour. The joint design should be studied also for the standard joints that are affected in compression by the same mechanism. Finally, the main runner joint should be analysed carefully in order to understand what is affecting its behaviour and eventually modify it.

Experimental results highlighted that the typology of runners can slightly affect the behaviour of the joints. In fact, the peculiar connection in Base15 joints allows smaller displacements in compression than Base24 joints. Moreover, the resistance in tension is related to the mechanical properties of transversal runners, as failure is reached with tearing of the slot.

Numerical models were developed and calibrated for each of the tested joints and a good correspondence with experimental results was achieved. Future development of the actual research can regard a detailed numerical modelling of complete suspended ceilings using the numerical models of joints, here presented. Moreover, the results can be applied to a parametric numerical investigation of the seismic behaviour of suspended ceilings, with sensitivity analyses of the influence of the performances of joints on the global behaviour of suspended ceilings.

3 Experimental evaluation of suspended ceiling behaviour

An innovative experimental setup was designed to overcome the limitations of shake-table tests, as described in Chapter 1. In particular, the aim was to realize inexpensive experimental tests, easy to be reproduced in many specimens. The idea was to apply to suspended ceilings the procedure of cyclic-quasi static tests, since it is a valid instrument commonly used to evaluate the seismic assessment of walls. Moreover, it leads to a seismic assessment that is independent of a specific input motion. In order to achieve this methodology, it was necessary to develop a test rig able to simulate the boundary conditions of a suspended ceiling, meaning the stiffness of the slab and the walls at perimeter.

The advantages of the system here proposed are also related to the possibility to adapt the protocol depending on the characteristics of the specimens and to easily evaluate the progression of damage, as the speed of the loading ramp is usually low. Moreover, the results will relate the resistance of the specimens to their displacement and they can be easily measured and elaborated. The experimental setup is flexible and can be easily adapt at different configurations.

The experimental campaign regarded a wide range of suspended ceilings, in order to investigate the type of damage they can incur into, the behaviour of the different categories and make a comparison between them. In particular, both metal and dry-wall ceilings are studied.

The main purpose of this work is to evaluate in general the failure mechanisms and specifically the performance of lateral bracings installed with seismic resistance purposes. In particular, in this chapter their contribution to the global resistance will be reported and their stiffness, ductility and energy dissipation will be investigated. Tests allow also to evaluate the performance of the ceiling components, e.g. hanger wires deformation and efficiency of connections.

3.1 Experimental setup

In the test rig (represented in Fig. 34 and Fig. 35) the soffit is simulated by a CLT (Cross Laminated Timber) slab, its dimensions in plane are 548x262 cm and it is 10 cm high. It is simply supported on two opposite sides to L-beams whose edge is 17 cm. The beams are rigidly connected to two columns each, realized with IPE 180 steel columns 300 cm-high. Columns are connected to steel plates at the ground with cylindrical hinges that allow the columns to rotate in the direction of application of the load. With the aim of keeping the columns vertical and to stiffen them in their weak axis, bracing chords are installed in the transversal direction. The perimeter boundaries are simulated with seven tubular beams with square section of 10 cm-edge and whose thickness is 3 mm. They are connected each other with hinge joints, realized with plates welded to the beams that can rotate around a metal cylinder, moreover friction is reduced interposing Teflon sheets between the plates (Fig. 38 a). The plane where tubulars are installed is parallel to the timber slab, forming two symmetrical squares whose dimensions are 240 cm. The aim is to test two adjacent portions of suspended ceilings, applying

symmetrical boundary conditions. The tubular beams are hung to the CLT slab with threaded rods connected at the two ends with spherical joints, realized with spherical washers that form a male to female type assembly (Fig. 37). With the aim of not damaging the slab and to facilitate the installation, brackets with omega shape are connected to the rods. The length of rods determines the void height (defined as the distance between the ceiling's intrados and the soffit), and it can be easily modified, cutting the rods or substituting them with longer ones. Four brackets are installed in the columns at the tubular beams height (Fig. 38 b), in order to direct the displacement of the suspended ceilings, but they are positioned at a small distance from the beams to avoid local friction forces.

Perimeter trims of suspended ceilings are fixed to timber beams with 5x3 cm section that are connected with screws to the tubular beams. This detail was achieved in order to avoid the damage of tubulars, as they represent the main structure of the rig. Suspended ceilings have all the details expected in a common installation, including hanger wires and seismic bracings. The hanger wires are attached with timber screws to the CLT system, while one seismic bracing is installed for each portion of ceiling and they are placed symmetrically in the loading direction. The seismic bracings are connected to the main runner and to the CLT slab with timber screws and their position was established in order to allow the right installation and avoid reciprocal interference.

Load is applied in displacement control to the tubular beam located in the middle of the two ceiling portions. The actuator (Fig. 36) is a screw mechanical actuator and it is installed between two UPN200 columns with some brackets, whose height can be modified according to the ceiling void height as the columns have holes positioned at regular spacing. Columns are connected to the CLT slab with a channel, in fact it allows the reaction forces to be transmitted to the ground. Columns are screwed to a metal base realized with brackets welded to a metal plate.

The design of the experimental setup also allows two different ceiling configurations depending on the presence of plane bracings realized with steel cords. The configuration chosen for this experimental campaign is defined "rigid floor" and it includes the installation of two symmetrical bracings in the ceilings' plane, connected to the tubular beams in proximity of the hinge joints. The plane bracings force the suspended ceiling portions to move simultaneously, allowing to evaluate the contribution of the seismic bracings. To realize the second configuration, called "deformable floor", the tubular beams in proximity to the IPE180 columns need to be fixed at them with a screwed connection through the brackets and plane bracings don't need to be installed. This configuration allows the central beam to move independently from the external beams, reproducing the case when two opposite walls have different displacements during earthquake motion. If these conditions are applied, the ceiling resistance to angular distortion can be evaluated.

The suspended ceiling displacement is measured in some keypoints (Fig. 39) with rotary potentiometers (numbered from 1 to 4) or linear potentiometers (5-6). Potentiometers from 1 to 3 measure the horizontal displacement of the ceiling, while potentiometer 4 measures the vertical displacement. The linear potentiometers 5 and 6 are located close to the seismic bracings in order to evaluate their relative displacements from the transversal tubulars. The applied force is measured by an S-cell load able to measure both tension and compression forces. The displacement of the actuator is measured by the potentiometer 7, in line with potentiometer 2.

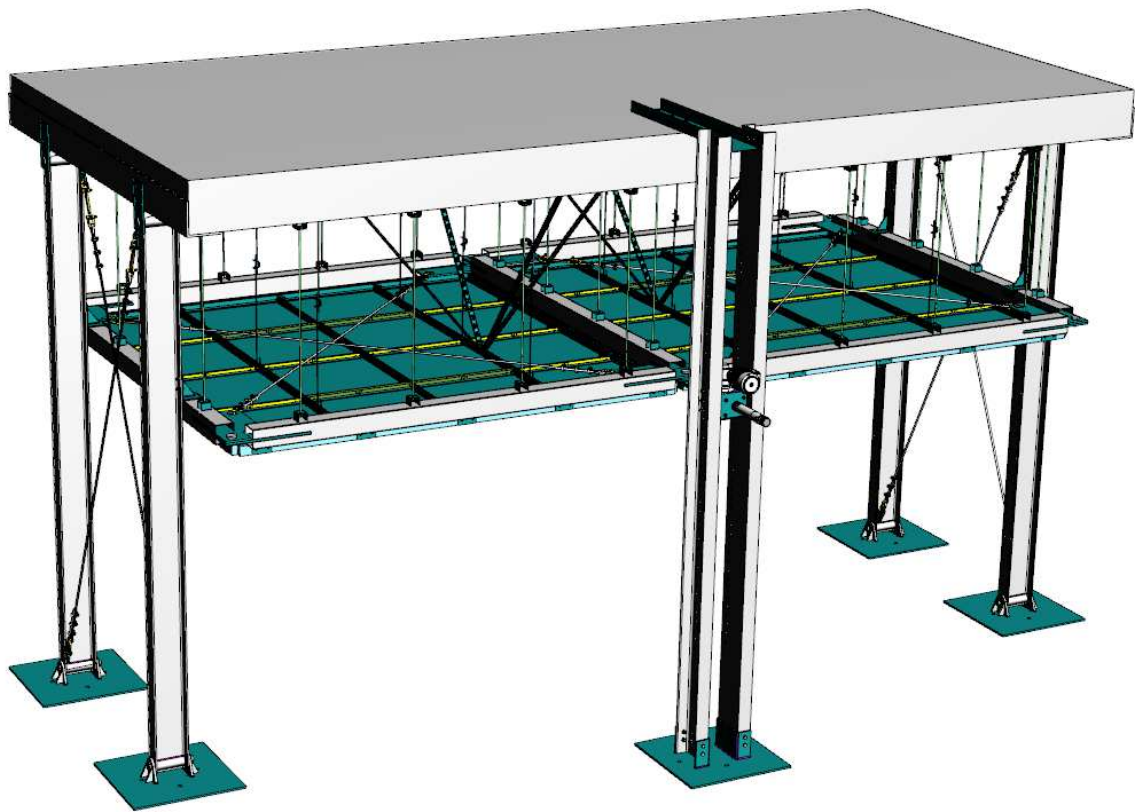


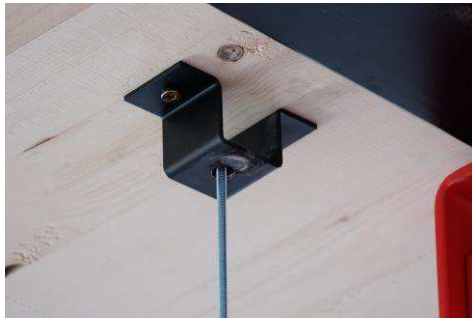
Fig. 34: Design of the experimental setup for cyclic quasi-static tests of suspended ceilings.



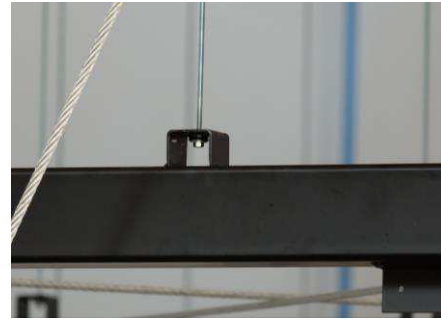
Fig. 35: Picture of the experimental setup.



Fig. 36: Mechanical actuator and load cell.

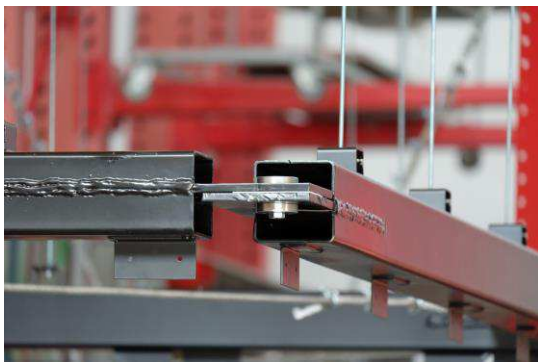


a)



b)

Fig. 37: Spherical joints connecting the threaded rod to the omega bracket. Detail of omega bracket a) screwed to the CLT slab at the top end, b) welded to the tubular beams at the bottom end.



a)



b)

Fig. 38: a): Hinge joints connecting two tubular beams; b): brackets that guide the displacement of tubular beams.

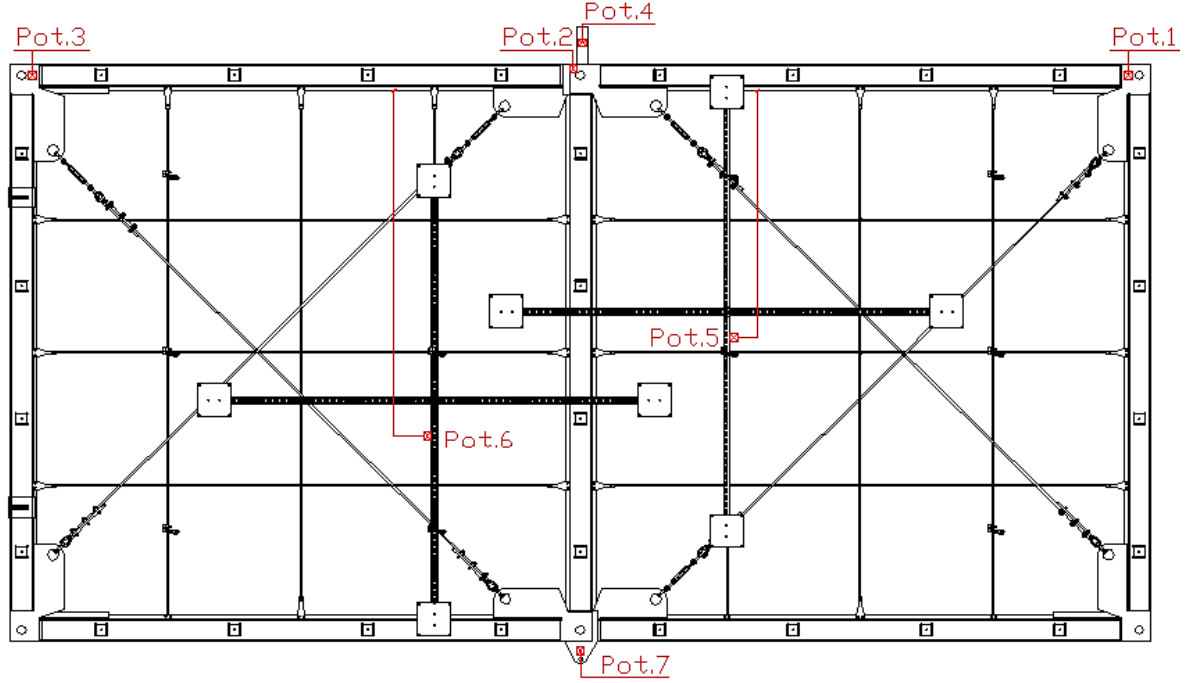


Fig. 39: Scheme of the position of potentiometers.

The experimental setup here presented can realize both monotonic and cyclic quasi-static tests. For each specimen, one monotonic and one cyclic quasi-static tests were performed.

In the monotonic test a loading ramp is applied in displacement control until reaching the collapse of specimens or the limit of the setup. In fact, the maximum displacement allowed by the experimental setup is determined by the stroke length of the actuator, that is ± 10 cm (in both directions). The monotonic tests allow to evaluate the yielding value of the specimen and the damage or collapse mechanisms.

To perform cyclic tests, the protocol defined in the FEMA 461 (Fig. 40) for cyclic quasi-static testing of non-structural components is adopted. The protocol requires the application of at least three increments before any damage of the specimen (identified with the amplitude Δ_0) and at least ten loading increments before reaching the collapse (Δ_m). For each loading amplitude, two cycles need to be performed and increments are defined with the following formulation:

$$a_{i+1} = 1.4a_i$$

Where a_i is the amplitude of the previous step. If at the end of the protocol, the specimen has not reached the collapse for the value of Δ_m previously defined, the cycles need to be applied with an amplitude constantly increased of the value $0.3\Delta_m$. Finally, the load is applied at a constant velocity of 18 mm/min.

As the monotonic tests didn't show a damage that could be uniquely defined, the choice of the amplitude of the cycles is related to the maximum stroke of the actuator, so the maximum amplitude is ± 10 cm and the amplitude of the other cycles are determined consequently. The protocol used for the tests is reported in Fig. 41.

The experimental setup was designed and built in order to be easily modified and improved. In particular, with the configuration that is here proposed, the vertical response of the system is not taken into account as the actuator can apply the force only in the ceiling plane. Moreover,

the actuator cannot apply impulsive actions or dynamic loads. These shortcomings can be overcome with the substitution of the actuator with another instrument able to apply different loading histories. Moreover, the height of the CLT slab was chosen to respond to both the need of a rigid floor and the possibility to excite it in the vertical direction by means of a vibrodyne installed on the top.

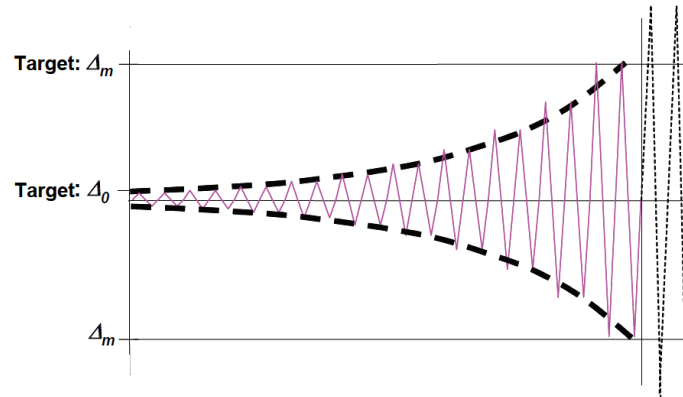


Fig. 40: FEMA461 protocol for cyclic quasi-static testing of non-structural components.

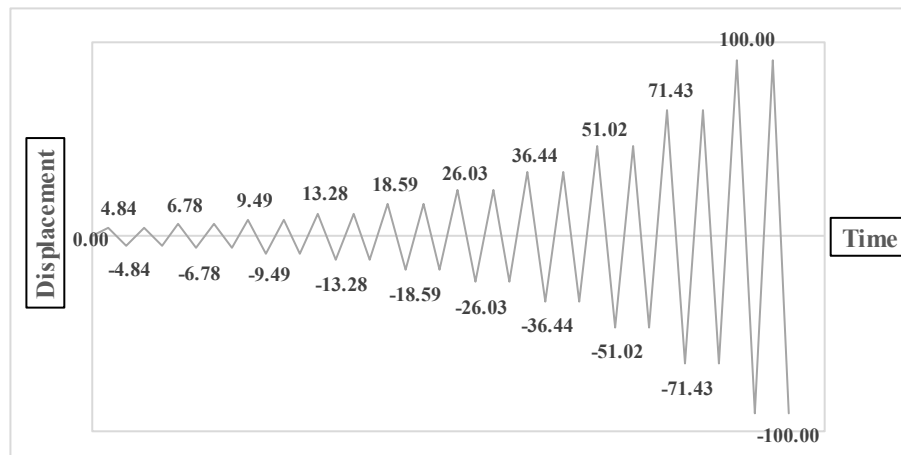


Fig. 41: Protocol for cyclic quasi-static testing of suspended ceilings.

3.2 Description of specimens

Specimens of representative typologies among metal and dry-wall suspended ceilings were chosen to be tested. Metal ceilings include the most common T-Grid ceilings with metal and lightweight gypsum tiles. The other two typologies of metal ceilings studied are realized with different types of steel profiles, resulting in a more articulated structure. Two typologies of dry-wall suspended ceilings, here named Plast Sing and Plast Doub, are investigated. They have the same structure made of steel channels and differ only for the application of single or double plasterboards.

The structure of the suspended ceilings here presented is realized with both primary and secondary runners. For all specimens, primary runners are installed in the loading direction, while secondary profiles are placed in the orthogonal direction. Finally, the height of the void was modified during the campaign, in order to evaluate the sensitivity of the results and damage mechanisms according to the void dimension. As presented in Table 18, the height of the void varies from 1.15 m for T-Grid specimens to 0.37 for Plast Sing Low and Plast Doub specimens. As consequence of the variation of the void height the inclination of the studs of the bracing system varies from 60° to 30° to the ceiling plane, in fact, the studs of the bracing systems are connected to the CLT soffit in the same points.

Table 18: Scheme of specimens tested

Test	Specimen	Void	Stud inclination	Loading
1	T-Grid with metal tiles	1.15 m	60°	Monotonic
2		1.15 m	60°	Cyclic
3	T-Grid with lightweight gypsum tiles	1.15 m	60°	Monotonic
4		1.15 m	60°	Cyclic
5	Metal ceiling A	0.71 m	45°	Monotonic
6		0.71 m	45°	Cyclic
7	Metal ceiling B	0.71 m	45°	Monotonic
8		0.71 m	45°	Cyclic
9	Plast Sing High	0.67 m	45°	Cyclic
10	Plast Sing Low	0.37 m	30°	Monotonic
11		0.37 m	30°	Cyclic
12	Plast Doub	0.37 m	30°	Cyclic

3.2.1 Metal ceilings

3.2.1.1 T-Grid with metal and lightweight gypsum tiles

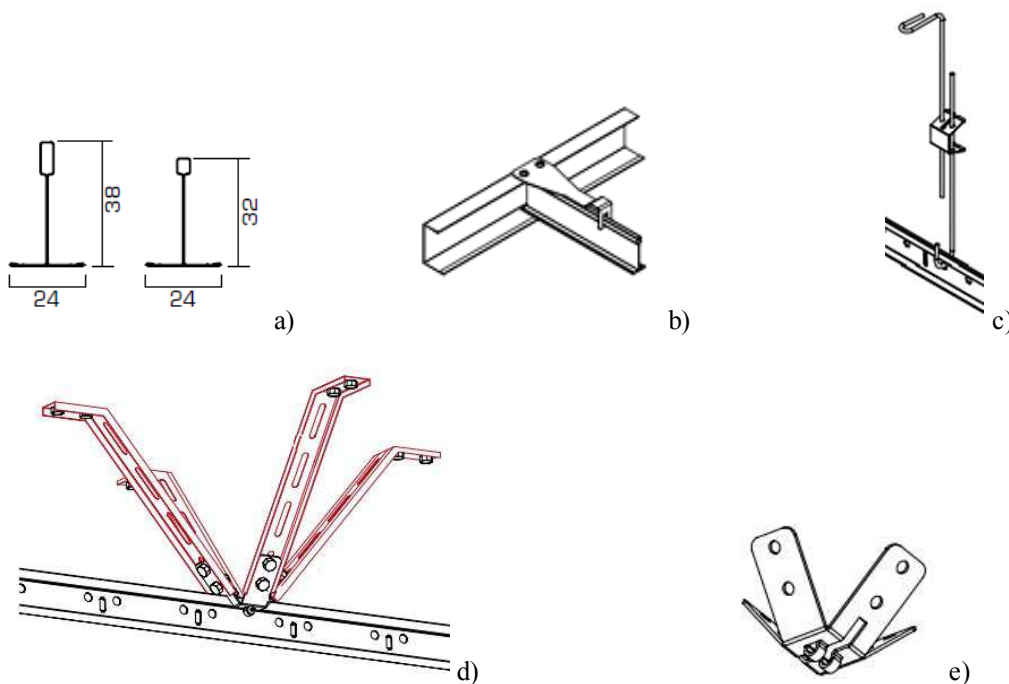
First, T-Grid specimens were tested. As illustrated in Fig. 42 a), they are realized with tee runners: the base is 24 mm wide and the height is 38 mm for main runners and 32 mm for cross runners. The seismic joints described in Chapter 2 are here installed; moreover, on two consecutive sides a clip is screwed to the perimeter trim (Fig. 42 b). The clip cannot realize a fixed connection of the runner to the perimeter but limit the displacement of the runner through friction phenomena. Primary runners are installed every 120 cm, while secondary runners are placed every 60 cm; they create a square mesh where tiles are set. The first and second specimens have metal tiles, whose weight is 5 kg/mq. The third and fourth specimens instead have plasterboard tiles, whose weight is 9.4 kg/mq. Primary runners are hung to the soffit with spring hanger rods (illustrated in Fig. 42 c), whose aim is to carry the weight of suspended ceilings. The seismic bracing (in Fig. 42 d) and f) is realized with an x-shape bracket at the base (Fig. 42 e) that is attached to the main runner. Four slotted studs are connected to the bracket with a screw and to the soffit with timber screws. The slope between the studs and the ceiling plane is 60 degrees.

3.2.1.2 Metal ceiling A

The structure of Metal ceiling A specimens is realized with steel U-channels, installed every 120 cm. A spring element (Fig. 43 b) connects primary and secondary runners, realized with tubular profiles placed every 60 cm. The clips illustrated in Fig. 43 a) fold the secondary runners and catch the metal tiles at their perimeter, in particular two clips are installed for each tile. Tiles are square with 60 cm-length side and their weight is 6.5 kg/mq. Moreover, hanger wires (illustrated in Fig. 43 c) are installed at maximum spacing of 120 cm. Finally, the seismic bracing (Fig. 43 d) and e) is similar to the one used in T-Grid specimens with the exception of the base of the bracket. In fact, the bracket has a wider base in order to be installed upon the U-channel runners, to which is connected with two adjacent screws passing through the runner. The seismic bracing studs are positioned at 45 degrees to the ceiling plane.

3.2.1.3 Metal ceiling B

Metal ceiling B is realized with U slotted primary runners, while secondary runners have Z-shape (Fig. 44 a) and b) and are connected to the primary runners with a particular bracket represented in Fig. 44 c). The bracket has two lateral notches and a slot to allow the tiles to be installed and fixed. The spacing of both primary and secondary runners is 120 cm, in order to accommodate tiles with 120 and 60 cm dimensions. Tiles measure 120x120 cm and their weight is 8 kg/mq. A particular hanging system is used and it is realized with a threaded rod connected to the primary runners with a bracket (a detail in Fig. 44 a). The seismic bracing is similar to the ones used in T-Grid ceilings and Metal ceiling A, except for the dimension of the base of the bracket that is dependent on the width of the primary runner where the bracket is installed. Also in Metal ceiling B, the bracket at the base of the seismic bracing is connected to the primary runner with two screws. As seismic bracings installed in Metal ceiling A, their studs are inclined at 45 degrees to the plane of the ceiling. The seismic bracing is illustrated in Fig. 44 d) and e).



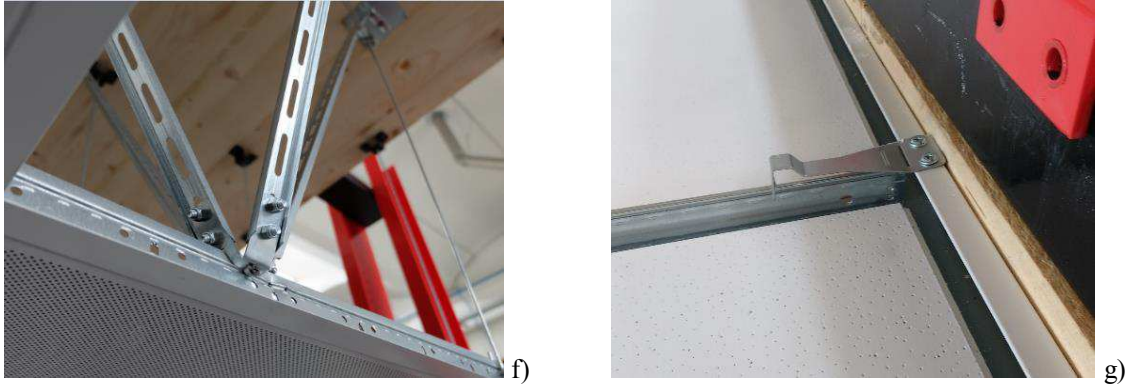


Fig. 42: T-Grid system: a) section of main and cross T-runners, b) perimeter connection, c) hanger rod, d) bracing system, e) detail of the seismic bracing bracket, f) picture of the seismic bracing installed in T-Grid with metal tiles, g) perimeter connection in T-Grid with gypsum tiles.

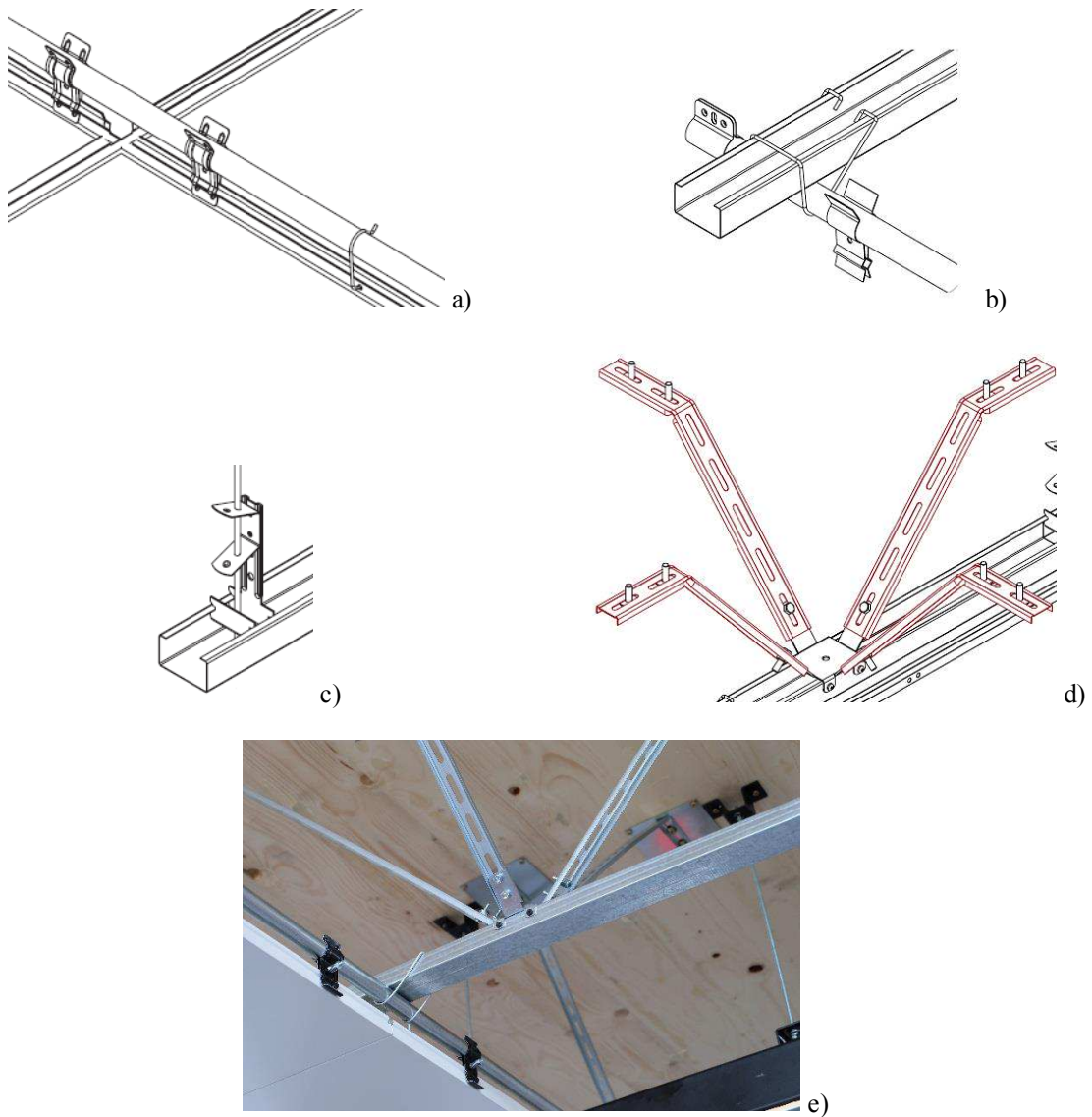


Fig. 43: Metal ceiling A system: a) detail of the clips holding the tiles, b) detail of the spring connecting primary and secondary runners, c) hanger wire, d) bracing system, e) picture of the seismic bracing system installed in a Metal ceiling A specimen.

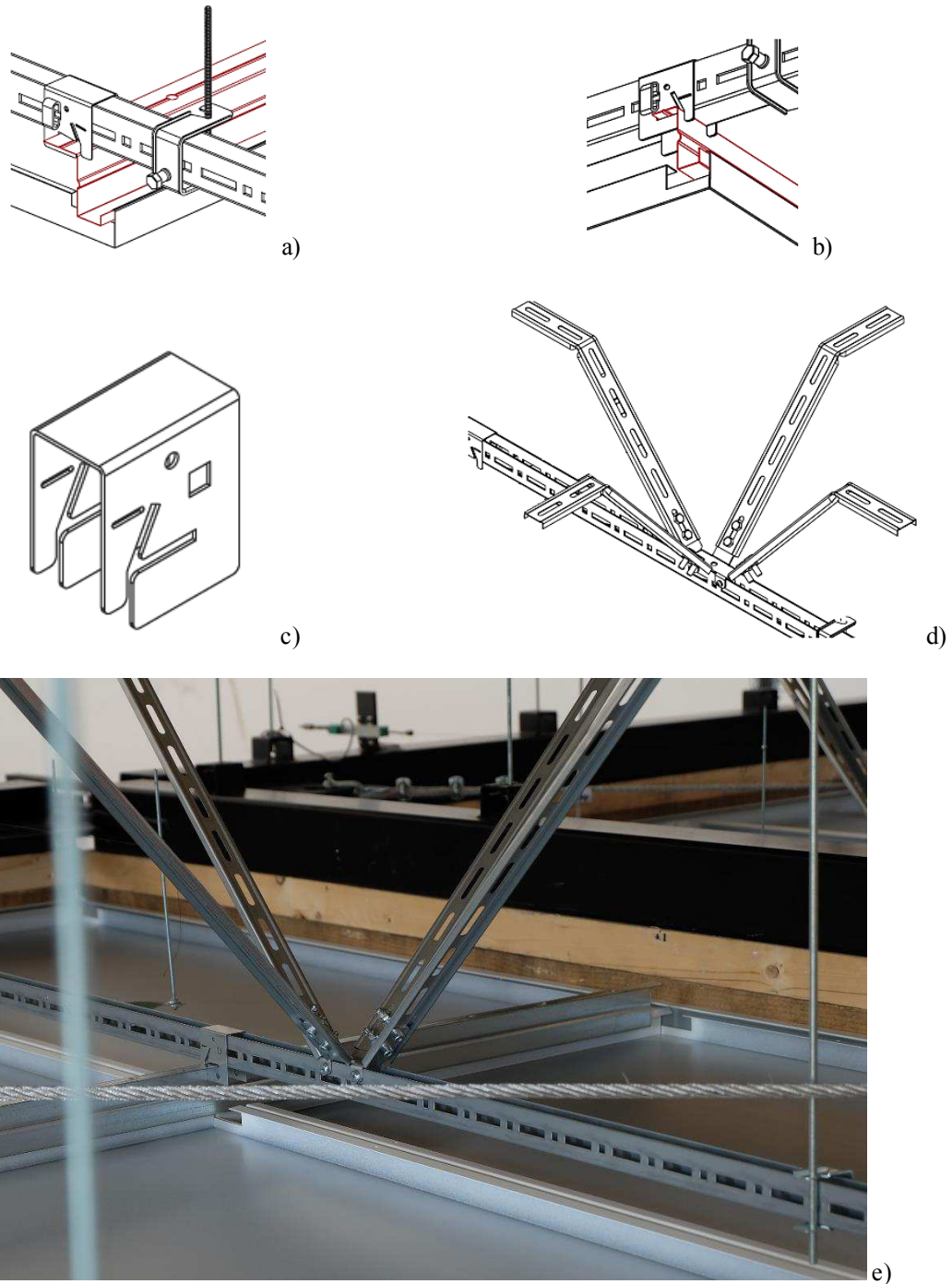


Fig. 44: Metal ceiling B: a) and b) connections between primary and secondary runners and tiles, c) bracket connecting primary and secondary runners, d) detail of the seismic bracing system, e) picture of the detail of the seismic bracing installed in a Metal ceiling B specimen.

3.2.2 Dry-wall ceilings

3.2.2.1 Plast Sing

Plast Sing is realized with primary and secondary runners, made of S235 steel C-channels. The section of runners is 48 mm large, 27 mm high and 0.6 mm thick. Primary runners are installed with 1 m of spacing while the spacing for secondary runners is 0.5 m (for a scheme see Fig. 45 c). Primary and secondary runners are connected with a bracket that is typically used for C-

channels in dry-wall systems and its thickness is 1 mm (Fig. 45 a). Spring hanger rods connect the primary runner to the soffit and they also have the purpose to carry the weight of the suspended ceiling. Hanger rods have 4mm-diameter and are installed with 1 m of spacing, they are fixed to the CLT slab with wood screws but the typology of screws used depends on the soffit to which the ceiling is installed. Secondary runners are screwed to a U-channel placed at the perimeter and screwed to the timber beams of the setup. The section of the U-channels is 28 mm high, 30 mm wide and 0.6 mm thick. A single plasterboard layer is screwed to the secondary runners in the orthogonal direction. The plasterboard is 13.5 mm thick and its weight is 9.3 kg/mq. The connection of the plasterboard to the runners is realized with self-drilling phosphate screws. Moreover, joints between plasterboards are sealed with a reinforcing tape and with gypsum-based stucco and the same detail is applied also to the perimeter edges. A seismic bracing identical to the ones previously described is installed on the primary runners. The base of the bracket is wider, in order to be installed upon the main runner (Fig. 45 b). Plast Sing specimens are tested with two different void height: 0.67 and 0.37 m, the specimen tested with the higher void height is defined Plast Sing High (Fig. 45 d), while the other Plast Sing Low. The inclination of the studs of the seismic bracing is 45° for Plast Sing High and 30° for Plast Sing Low.

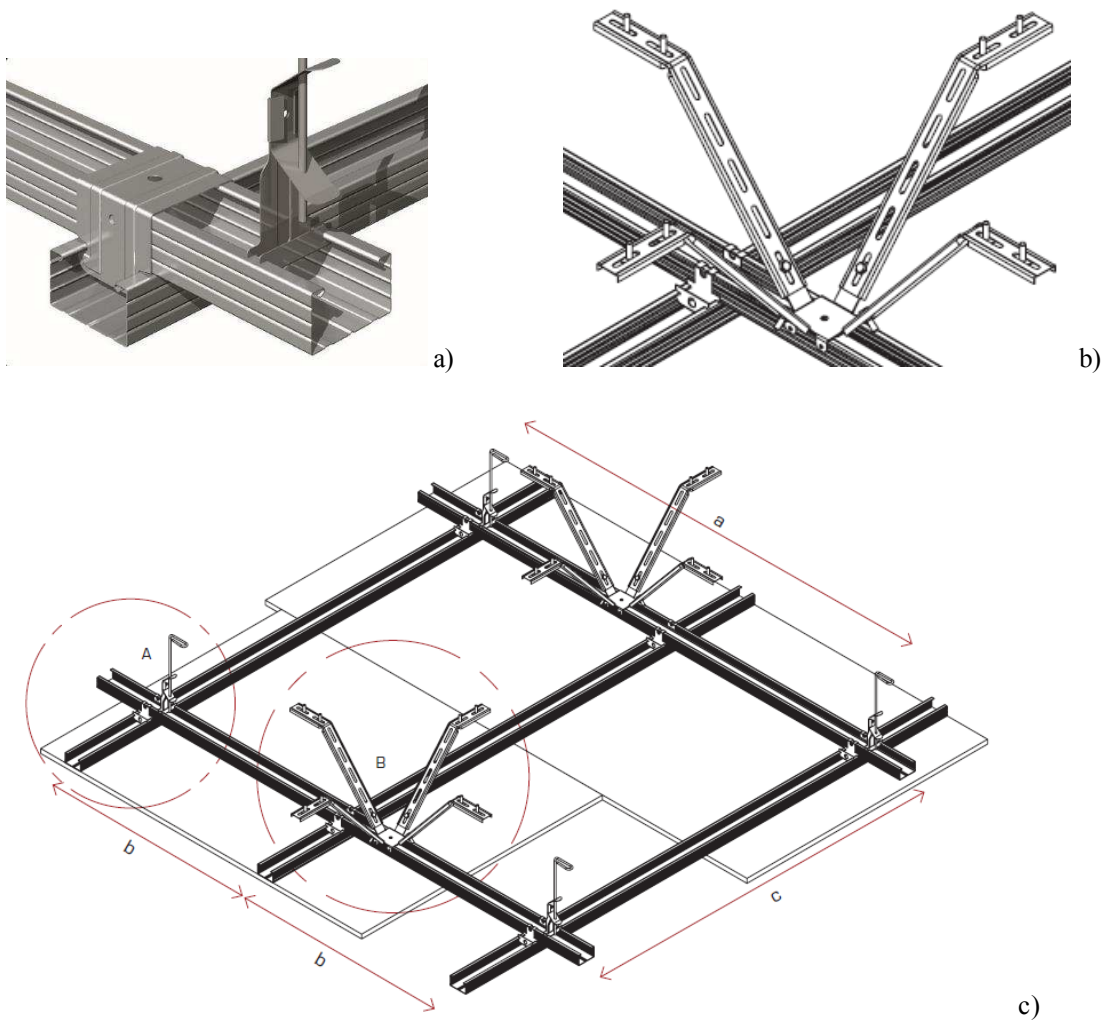
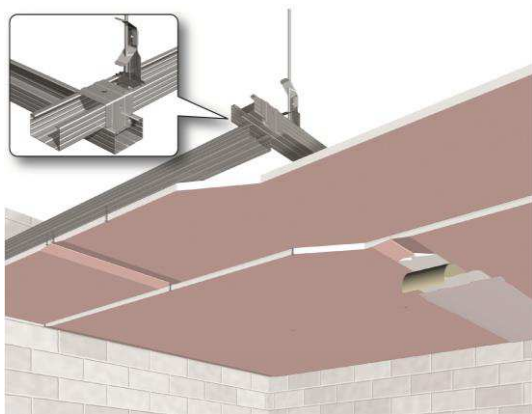




Fig. 45: Plast Sing system: a) detail of the connection between primary and secondary runners and hanger rods, b) detail of the seismic bracing system, c) placement of the seismic bracings and the hanger rods, d) specimen of Sing Plat High.

3.2.2.2 Plast Doub

Plast Doub is realized with the same metal structure as Plast Sing, e.g. runners, hanger rods and seismic bracing. The difference is that in Plast Doub there are two plasterboards screwed to the secondary runner, whose thickness is 15 mm and the weight is 13.6 kg/mq for each plasterboard (27.2 kg/mq in total). As illustrated in Fig. 46 a) and b), the second layer of plasterboards is installed orthogonally to the first one. The inclination of the studs of the seismic bracing is 30° to the ceiling plane (Fig. 46 c) and d). A specimen of Doub Plast is reported in Fig. 46 b).



a)



b)



c)



d)

Fig. 46: Plast Doub system: a) placement of the runners and the plasterboards and details of the bracket and the hanger rods, b) picture of the installation of primary and secondary runners, c) installation of the seismic bracing system, d) picture of a specimen.

3.3 Experimental results

For every typology of suspended ceiling, the experimental results of monotonic and cyclic tests are reported. Graphs represent displacement as a function of the force applied. Displacement plotted in the graphs is that measured from the potentiometer 7 (positioned in proximity to the load cell), unless otherwise noticed. Force is that measured by the load cell, i.e. it takes into account the resistance of the test rig. Therefore preliminary tests were conducted on the test rig without the suspended ceilings installed in it: a complete cycle was reproduced, applying to the setup the maximum displacement allowed in both directions. This procedure permitted to evaluate the effects of the resistance of the threaded rods, connecting the tubular beams to the CLT ceiling and the friction between the lateral tubular beams and the columns. Moreover, as the metal structure containing the suspended ceilings is hung to the soffit, its behaviour is similar to a pendulum and the change of direction of the force applied generates additional inertia forces that don't have to be considered in the value of resistance analysed. So, the resistance of the test rig is deducted from the value measured by the load cell and then divided by the number of seismic bracings installed. Thus, the force plotted in the graphs relates the resistance of a single seismic bracing to the displacement of the setup.

The results obtained from the tests have been elaborated to obtain the mechanical parameters and dissipative characteristics of the system studied. In particular, from the results of each cyclic tests an envelope curve was defined with the analytical formulation proposed by Foschi and Bonac (1977). Then, an elasto-plastic function was defined through bilinearization of the envelope, with the aim of evaluating the yielding and ductility values. For the metal ceilings, an elasto-perfectly plastic function was defined according to the procedure indicated in NTC08 (§ C7.3.4.1). As represented in Fig. 47, the elastic branch of the bilinear crosses the envelope for $0.6F_{bu}^*$, where F_{bu}^* is the value of the maximum resistance in the envelope. For an envelope curve with plastic behaviour with hardening, the plastic branch of the bilinear ends when it reaches the maximum resistance. In the situation where the envelope shows degradation, the final displacement of the bilinear corresponds to $0.85F_{bu}^*$. The yielding point is determined imposing the equivalence of the areas under both envelope and bilinear curves. For the specimens of drywall suspended ceilings, another equivalent-energy method is used, as the bilinear function is elasto-plastic with hardening (Pozza et al., 2015). The ductility value μ is defined as the ratio between the final displacement d_u^* and the yielding displacement d_y .

For each increment of displacement applied to the system during cycling tests, two identical cycles are performed. This procedure allows to define the value of strength degradation, calculated as the difference between the peak resistance between the first and the second cycle. However, in all tests, the value of strength degradation of specimens was negligible and then the values are not reported.

Equation (1) was used to calculate the value of viscous damping ratio v_{eq} . It is an adimensional parameter that describes the hysteresis damping properties and that is proportional to the ratio of the energy dissipated for half-cycle (E_d) and the potential energy (E_p):

$$v_{eq} = \frac{E_d}{2\pi \cdot E_p} \quad (1)$$

As indicated in Fig. 48, the energy dissipated in half-cycle corresponds to the inner area of half-experimental cycle, while the potential energy is the area under the triangle that has as height the maximum resistance of the cycle considered and as base the correspondent

displacement. For the metal ceilings tested, the viscous damping ratio is calculated for the second cycle for each amplitude in the pull loading. For the drywall ceilings, as the results are strongly asymmetrical for some tests, the cycles are considered only during push loading.

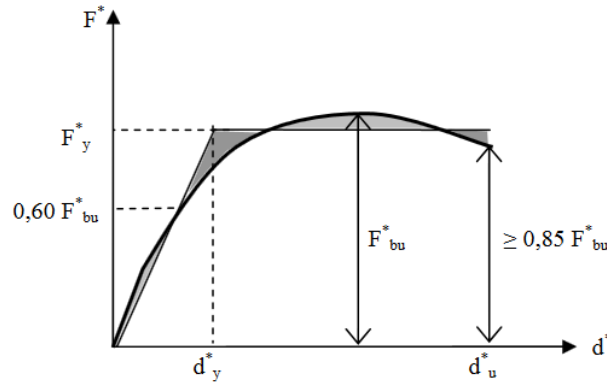


Fig. 47: System and bilinear equivalent function according to NTC08 (§ C7.3.4.1)

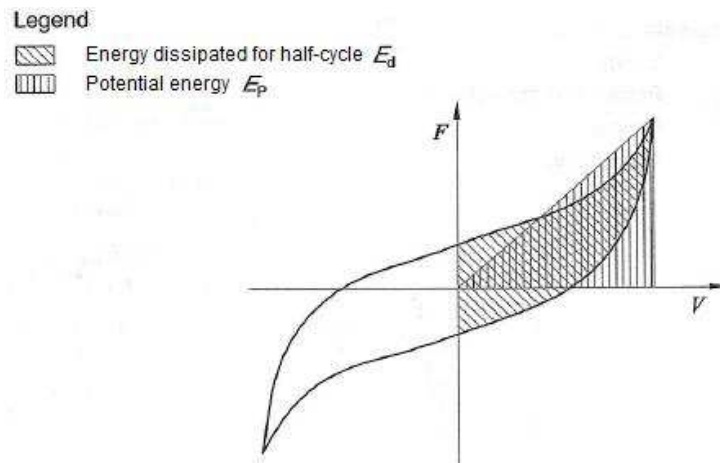


Fig. 48: Definition of the viscous damping ratio value.

3.3.1 T-Grid with metal tiles

In Fig. 49, the results obtained from the T-Grid specimens with metal tiles are reported. The displacement indicated in the graph corresponds to the potentiometer 2, since the potentiometer 7 had problems in transmitting the signal during the cyclic test. The monotonic test was necessary to evaluate the yielding value and the maximum resistance obtained from the specimens. Moreover, it gave information about the progression of damage (even if in only one direction) and the final damaged configuration. The cyclic test confirmed the damage mechanisms observed in the monotonic test. Furthermore, it showed the reaction of ceiling components to the change of direction of the force applied and to the repetition of cycles.

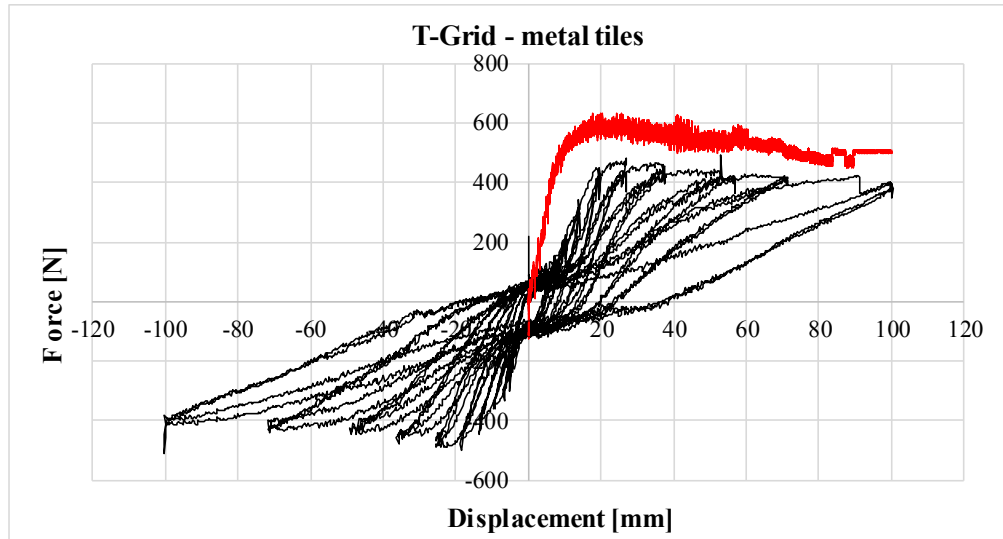


Fig. 49: Force-displacement graph of specimen 1 (monotonic - in red) and specimen 2 (cyclic - in black), (displacement refers to potentiometer 2).

From graphs reported in Fig. 49, it can be noticed that monotonic and cyclic tests have similar results and the behaviour of specimens is elasto-plastic with degradation after the peak of resistance. Results from monotonic test are slightly higher than cyclic test results but this difference can be attributed to the repetition of loading during cyclic tests. Damage is similar in both tests and it is located in the slotted studs of seismic bracings.

Specimen 1 showed elastic behaviour until reaching 5 mm of displacement, that corresponds to the starting of the buckling phenomenon that affected the stud subjected to compression forces (in the loading direction). After the initial elastic behaviour, the curve decreases its gradient reaching the maximum resistance of 600 N at 20 mm. The final resistance is 500 N. The buckling of the first stud caused the uplift of the base of the seismic bracing and consequently the compression of the studs orthogonal to the loading direction. Accordingly, three studs are compressed and one stud is subjected to tensile stress. This phenomenon is related to the degradation of the resistance after 20 mm. In particular, the studs pull the base of the seismic bracings with the consequent uplift of the runner to which are connected. The lift of the runner is not distributed equally in all its lengths, as one end of the runner is blocked at the perimeter with the connection described in Fig. 42 b). In particular, the deformation on the main runner is located in the area in correspondence to the attachment of the seismic bracing (Fig. 50 b). This damage confirmed that the connections of the seismic bracings to the main runners are over resistant than the buckling load. The tiles did not fall but slightly moved from their initial position, in consequence of the damage on the main runner. Details of the damage are illustrated in Fig. 50.

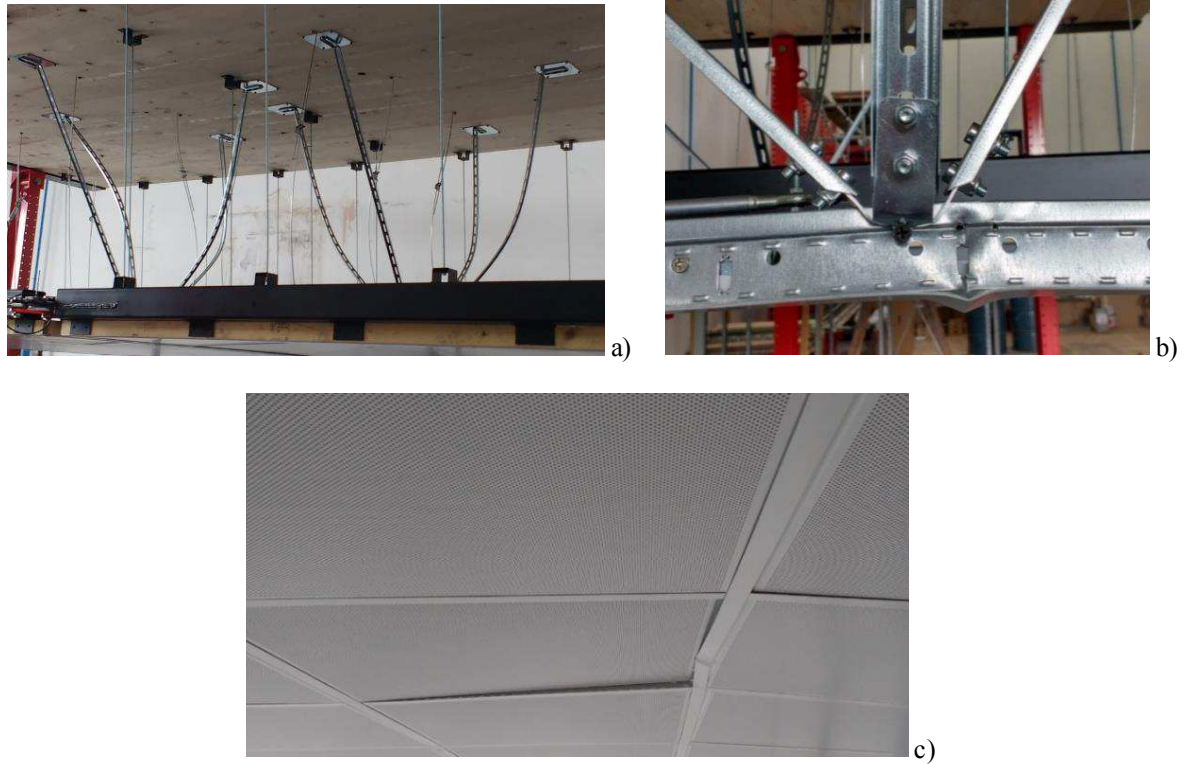


Fig. 50: Damage mechanism of T-Grid with metal tiles after monotonic test. a) buckling of the studs under compression, b) Deformation of the main runner in correspondence to the attachment point of the seismic bracing, c) deformation of the main runner and slight dislodgment of tiles

The cyclic test on T-Grid with metal tiles showed the same behaviour of the monotonic test and the specimen reached the maximum resistance of 450 N. Similar damage mechanisms to the monotonic test were developed. The only difference is the buckling of the other stud in the direction of the force applied, in consequence to the change of direction of the load during the cycles. In particular, the studs in the loading direction are consecutively under compression and tension. As in T-Grid specimens the bracket at the base of the seismic bracing is connected to the runner only with one screw, the alternate loading caused the rotation of the bracket around the connection point. Moreover, part of the displacement applied is transferred to the bracket at the base, but when the rotation is stopped by the web of the runner, the displacement is in part transferred to the runner that slightly moves and impacts at the perimeter. Details of the damage are reported in Fig. 51.

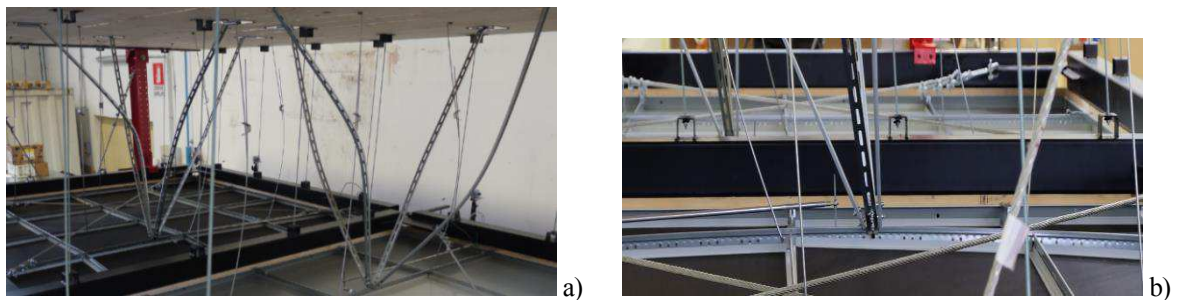




Fig. 51: Damage mechanism of T-Grid with Metal tiles after cyclic test: a) buckling phenomena on the slotted studs of seismic bracings, b) lift of the main runner and rotation of the bracket of the seismic bracing, c) uplift and slight dislodgement of tiles in the area close to the seismic bracing.

In Fig. 52, the envelope and bilinear curves elaborated from the experimental results are reported. It can be noticed that the results of the cyclic test on the specimen 2 have symmetrical behaviour in both push and pull loading. In Table 19 the characteristic values obtained from the envelope and bilinear curve are indicated. In Table 20, the values of viscous damping ratio are reported and it can be noticed that they are almost constant for all cycles.

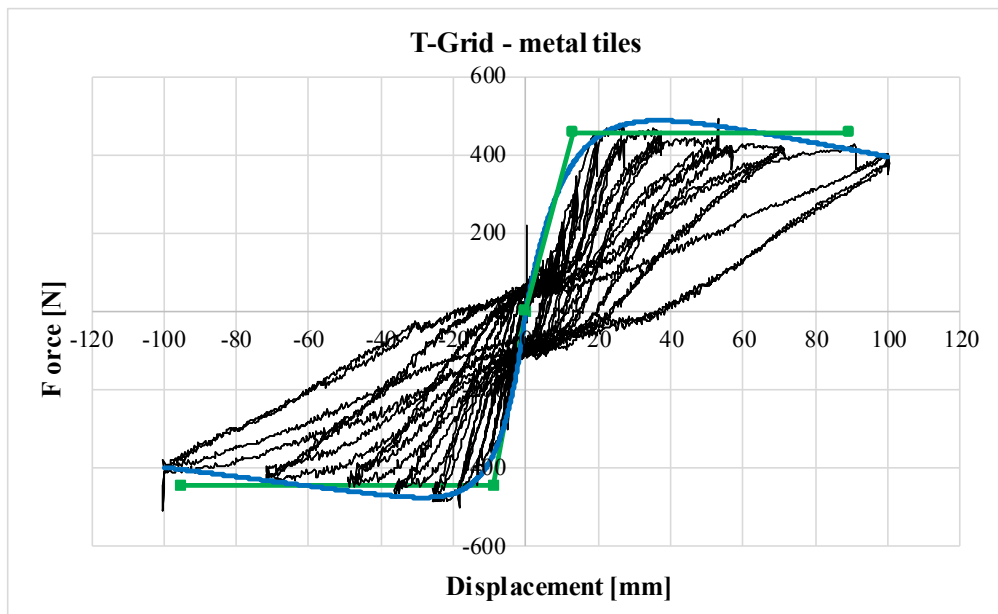


Fig. 52: Envelope curve and bilinear function for the specimen T-Grid with Metal tiles.

Table 19: Analysis of the results from cyclic test on T-Grid with metal tiles specimen.

Parameters		Push loading	Pull loading
Maximum displacement	$\delta_{\max}$ (mm)	89.50	-95.50
Ultimate force	F_{ult} (N)	454.12	-444.45
Maximum force	$F_{\max}$ (mm)	491.25	-512.30
Yielding displacement	δ_y (mm)	12.92	-8.88
Yielding force	F_y (N)	454.12	-444.45
Ductility	$\mu = \delta_u / \delta_y$	6.93	10.76

Table 20: Viscous damping ratio of T-Grid with Metal tiles specimen.

Cycle amplitude (mm)	Viscous damping ratio v_{eq} (%)
4.84	6.90
6.78	5.79
9.49	6.06
13.28	5.17
18.59	5.74
26.03	6.42
36.44	6.34
51.02	5.99
71.43	6.51
100.00	7.09

3.3.2 T-Grid with lightweight gypsum tiles

The monotonic test on T-Grid with lightweight gypsum tiles gave similar results than the specimen with metal tiles, in fact they differ only for the tiles installed. With the aim of reaching the weight of 9 kg/m^2 , two identical boards made of lightweight gypsum were installed to create the tile. The results obtained from monotonic and cyclic tests are reported in Fig. 53.

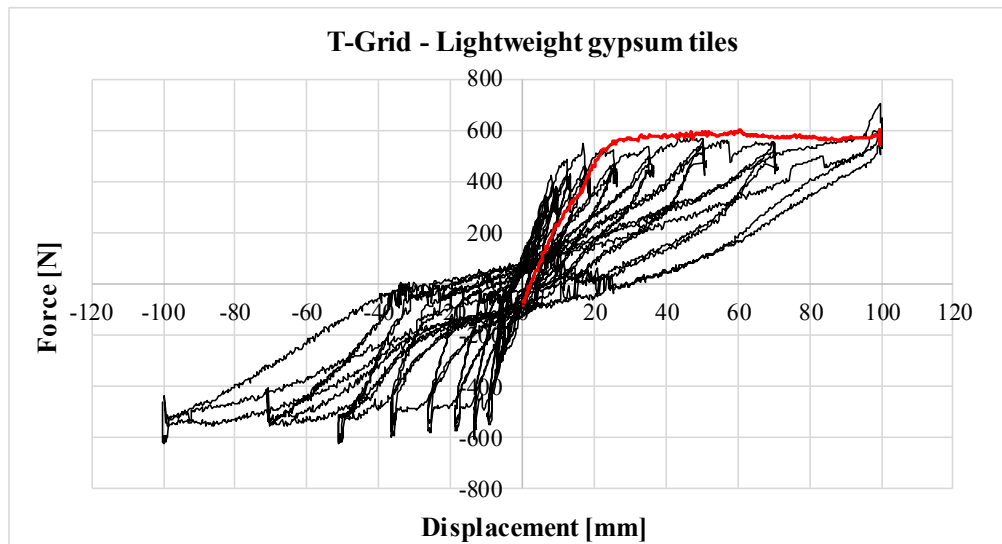


Fig. 53: Force-displacement graph of specimen 3 (in red) and specimen 4 (in black), referred to potentiometer 7

The monotonic test shows an elasto-plastic behaviour with less degradation than the results from the specimen of T-Grid with metal tiles. The maximum resistance of each seismic bracing is 600 N. The different plastic behaviour can be related to the weight of the panels. In fact, the mass contrast the deformation of the main runner where the seismic bracing is connected and consequently reduces the strength degradation caused by buckling of the studs. As it can be noticed in Fig. 53, monotonic and cyclic tests have similar results, in particular regarding the plastic behaviour, while the cyclic test shows an higher elastic stiffness.

The damage mechanisms reported by both tests are similar to the damage of T-Grid with metal tiles: buckling of the studs under compression (Fig. 54 a), uplift of the main runner in consequence to the buckling of the studs, deformation of the flange in correspondence to the attachment of the seismic bracing and finally rotation of the bracket of the seismic bracing (Fig. 54 b). In both tests, the deformation of main runners and the limited deformability of gypsum

tiles caused the slight dislodgment of the tiles and the partial fall from its seat (Fig. 54 c) and d). At the end of the tests any tile falls down.

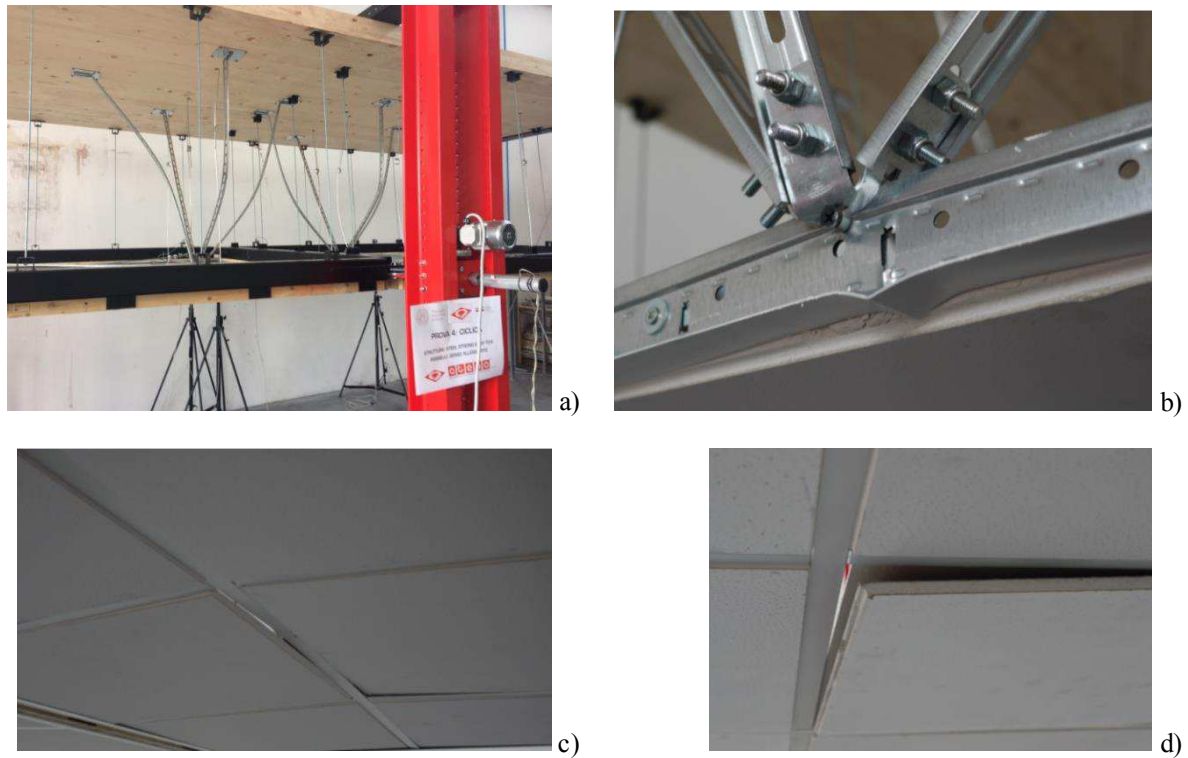


Fig. 54: Damage mechanism of T-Grid with lightweight gypsum tiles: a) buckling of the slotted studs of the bracing system, b) deformation of the main runner in correspondence to the attachment of the seismic bracing, c) dislodgement of tiles after the monotonic test, d) partial collapse of one tile.

In Fig. 53, the envelope and bilinear curves elaborated from the experimental results are reported. The results of the cyclic test of the specimen of T-Grid with lightweight gypsum tiles are generally symmetrical in push and pull loading. The mechanical parameters elaborated are reported in Table 21. In Table 22, the viscous damping ratio values are indicated: it can be noticed that it is high for the first cycles, and then, it decreases in the following cycles and remains constant.

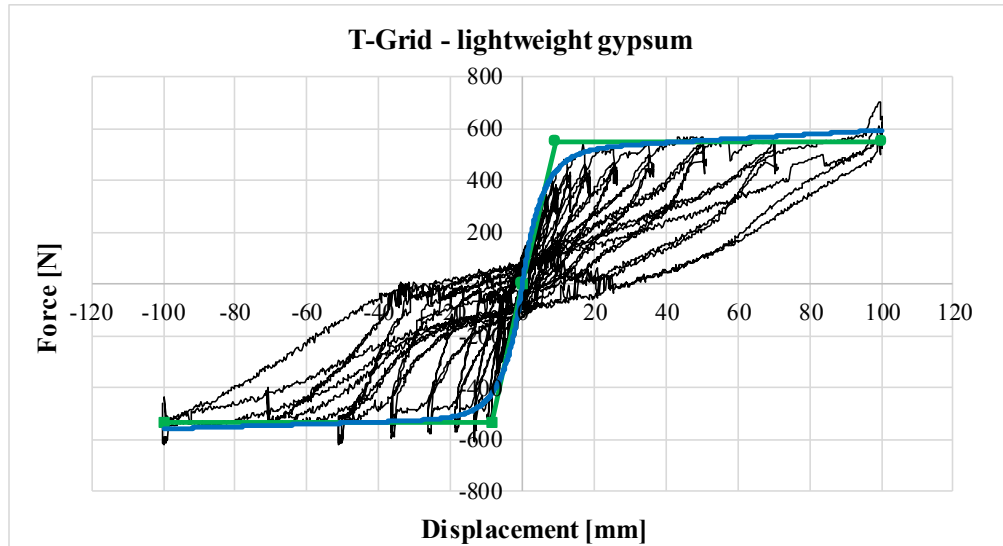


Fig. 55: Envelope curve and bilinear function for the specimen T-Grid with lightweight gypsum tiles.

Table 21: Analysis of the results from cyclic test on T-Grid with lightweight gypsum tiles specimen.

Parameters		Push loading	Pull loading
Maximum displacement	δ_{max} (mm)	100.00	100.00
Ultimate force	F_{ult} (N)	560.00	-560.00
Maximum force	F_{max} (mm)	705.40	-619.89
Yielding displacement	δ_y (mm)	9.24	-8.38
Yielding force	F_y (N)	552.39	-532.26
Ductility	$\mu = \delta_u / \delta_y$	10.83	11.93

Table 22: Viscous damping ratio of T-Grid with lightweight gypsum tiles specimen.

Cycle amplitude (mm)	Viscous damping ratio v_{eq} (%)
4.84	29.48
6.78	18.76
9.49	7.95
13.28	6.45
18.59	7.60
26.03	7.36
36.44	8.27
51.02	6.45
71.43	7.94

3.3.3 Metal ceiling A

Tests conducted on Metal ceiling A specimens are realized with the void value of 0.71 m, in order to evaluate the resistance of the system with a lower value of effective length of the studs than T-Grid specimens.

During the tests, a malfunctioning that affected the experimental setup prevented the free movement of the lateral tubular beams for high displacements. In fact, this phenomenon was caused by the brackets installed in the columns, where a concentration of friction forces was noticed. Then, it caused an increment of forces on the plane bracings that caused their deformation and consequently their inefficiency, so that the “rigid floor” configuration couldn’t

be maintained and different displacements of the lateral tubular beams were also measured. Since this phenomenon was effective after values of displacement higher than 4 cm, the results are reported until the system reached this value. In particular, the value of displacement plotted is a mean value of the displacements of potentiometers 1, 2, 3 and 7. In Fig. 56, the results reported are referred to maximum amplitude of 36.44 mm. This limitation can be considered acceptable, as it is a higher value than the maximum displacements usually considered for a suspended ceiling.

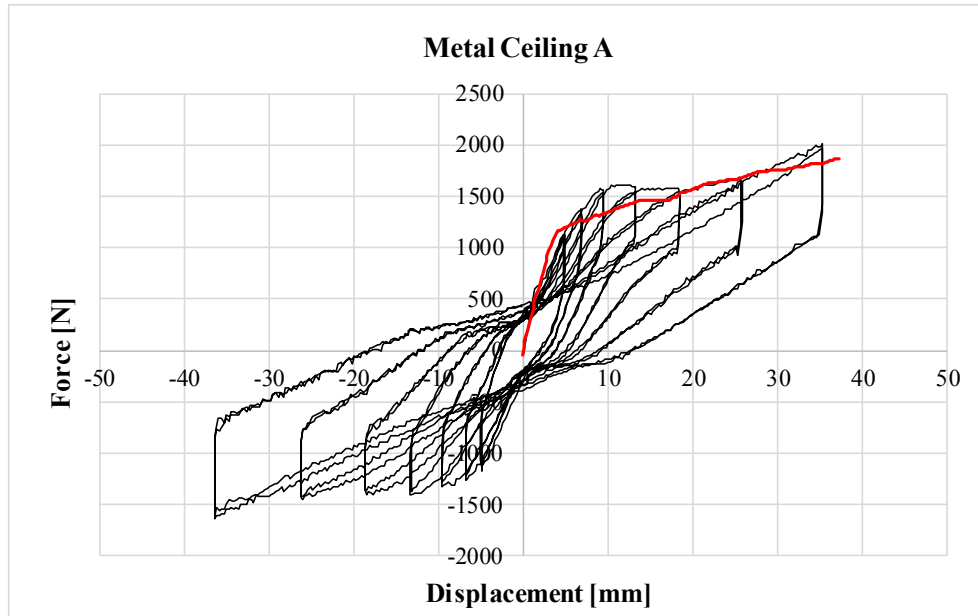


Fig. 56: Force-displacement graph of specimen 5 (in red) and specimen 6 (in black), referred to mean value of displacements of potentiometer 1, 2, 3 and 7.

The monotonic test outlined that the behaviour of the specimen is elasto-plastic with hardening. In particular, the yielding value was obtained for each seismic bracing at 1200 N and 4 mm of displacement. The hardening slope allows the specimen to reach 1900 N at 36.44 mm of displacement. The hardening behaviour is caused by the connection of the seismic bracing to the main runner. In fact, differently from T-Grid specimens, the main runner is a C-channel and the seismic bracing is connected to it with four screws that prevent the rotation of the bracket at the base (as seen in T-Grid specimens) and induce higher bending forces on the main runner.

Monotonic and cyclic tests have similar behaviour, as it can be noticed in Fig. 56. In both tests, buckling of the slotted studs was shown. The deformation of the studs caused the uplift and damage of the primary runner in correspondence to the point of attachment of the seismic bracing. The bending of the primary runner caused the detachment of the panels from secondary runners, that can be seen in the area of the attachment of the seismic bracing (Fig. 57 b). At the end of the cyclic test, the bracket that connects the seismic bracing to the main runner didn't show any bearing of the metal sheet in correspondence to the screw attachment, but damage to the web of the C-channel was shown, as consequence of the compression and local bending actions (Fig. 58 b). Details of the damage reported after monotonic and cyclic tests are illustrated in Fig. 57 and Fig. 58.

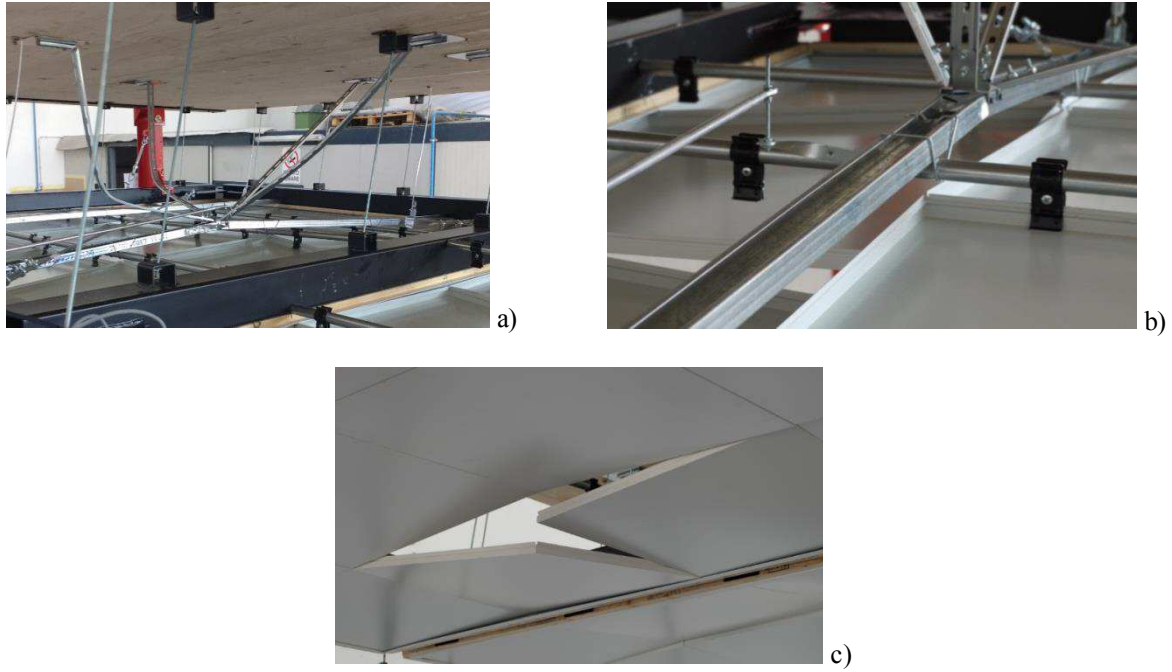


Fig. 57: Damage of Metal ceiling A specimens during monotonic test: a) buckling of the studs of the seismic bracing, b) bending of the main runner and detachment of the tiles, c) partial collapse of tiles.

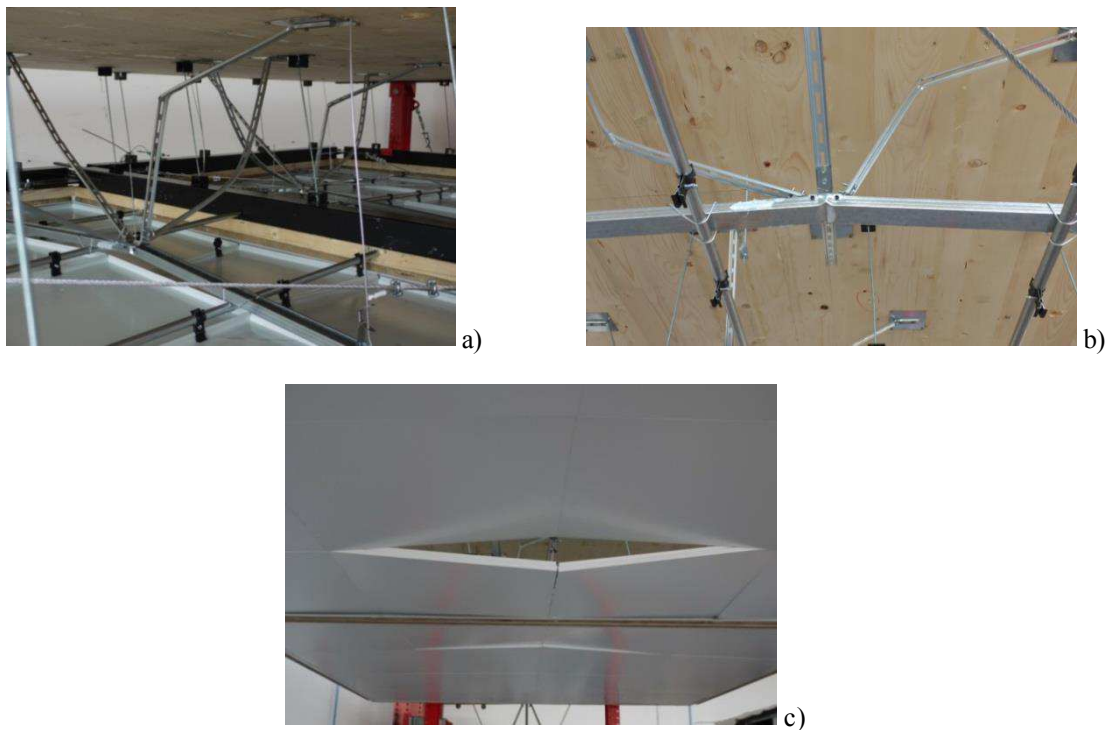


Fig. 58: Damage of Metal ceiling A specimens during cyclic test: a) buckling of the studs of the seismic bracing, b) deformation of the main runner at the end of the cyclic test, c) partial collapse of tiles.

In Fig. 59, the envelope and bilinear curves elaborated from the experimental results are reported. The results of the cyclic test on the Metal ceiling A specimen are symmetrical in push and pull loading only for the initial elastic behaviour. In Table 23, the mechanical parameters, elaborated from the envelope and bilinear curve, are reported. In Table 24, the values of viscous

damping ratio are indicated and it can be observed that it maintains a constant value in all the cycles considered.

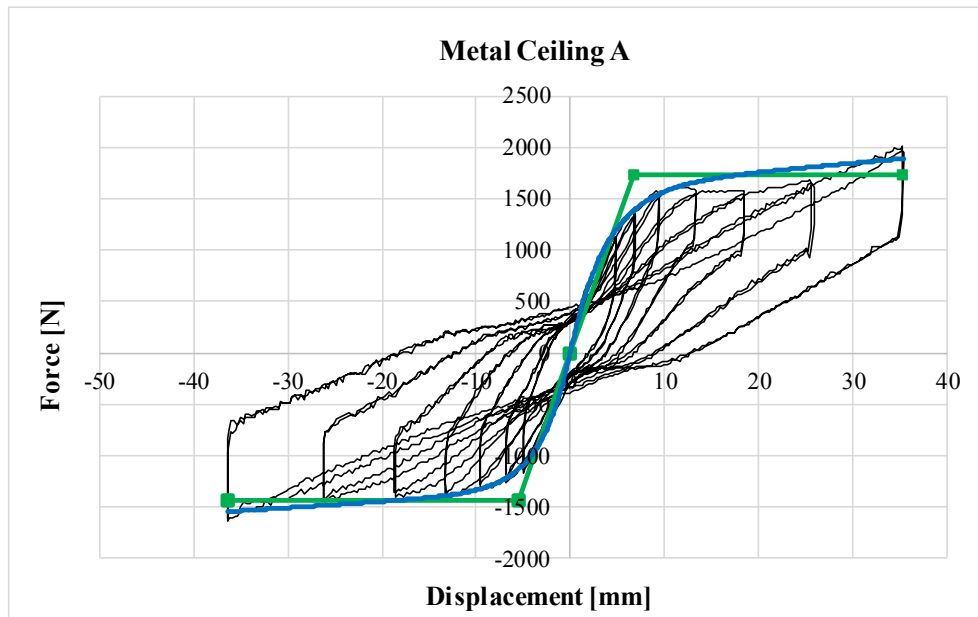


Fig. 59: Envelope curve and bilinear function for the specimen Metal ceiling A.

Table 23: Analysis of the results from cyclic test on Metal ceiling A specimen.

Parameters		Push loading	Pull loading
Maximum displacement	$\delta_{\max}$ (mm)	35.40	-36.50
Ultimate force	F_{ult} (N)	1890.14	-1427.87
Maximum force	$F_{\max}$ (mm)	2012.03	-1636.10
Yielding displacement	δ_y (mm)	6.72	-5.51
Yielding force	F_y (N)	1734.03	-1427.87
Ductility	$\mu = \delta_u / \delta_y$	5.27	6.62

Table 24: Viscous damping ratio of Metal ceiling A specimen.

Cycle amplitude (mm)	Viscous damping ratio v_{eq} (%)
4.84	16.86
6.78	15.75
9.49	15.75
13.28	16.25
18.59	15.88
26.03	17.71
36.44	17.57

3.3.4 Metal ceiling B

Fig. 60 a) and b) show the force values of the Metal ceiling B specimens in function of the mean displacement, obtained from monotonic and cyclic tests. A comparison from the two graphs highlights a great difference between the two responses. The differences are only caused by the different behaviour of the two specimens, as no problem in the experimental setup was noticed during tests. The monotonic test outlined a higher initial stiffness with a strong

degradation after that, and this phenomenon is repeated at 40 mm of displacement. Otherwise, the cyclic test has low value of initial stiffness and only after 20 mm of displacement the stiffness increases, reaching a final resistance value of 1200 N.

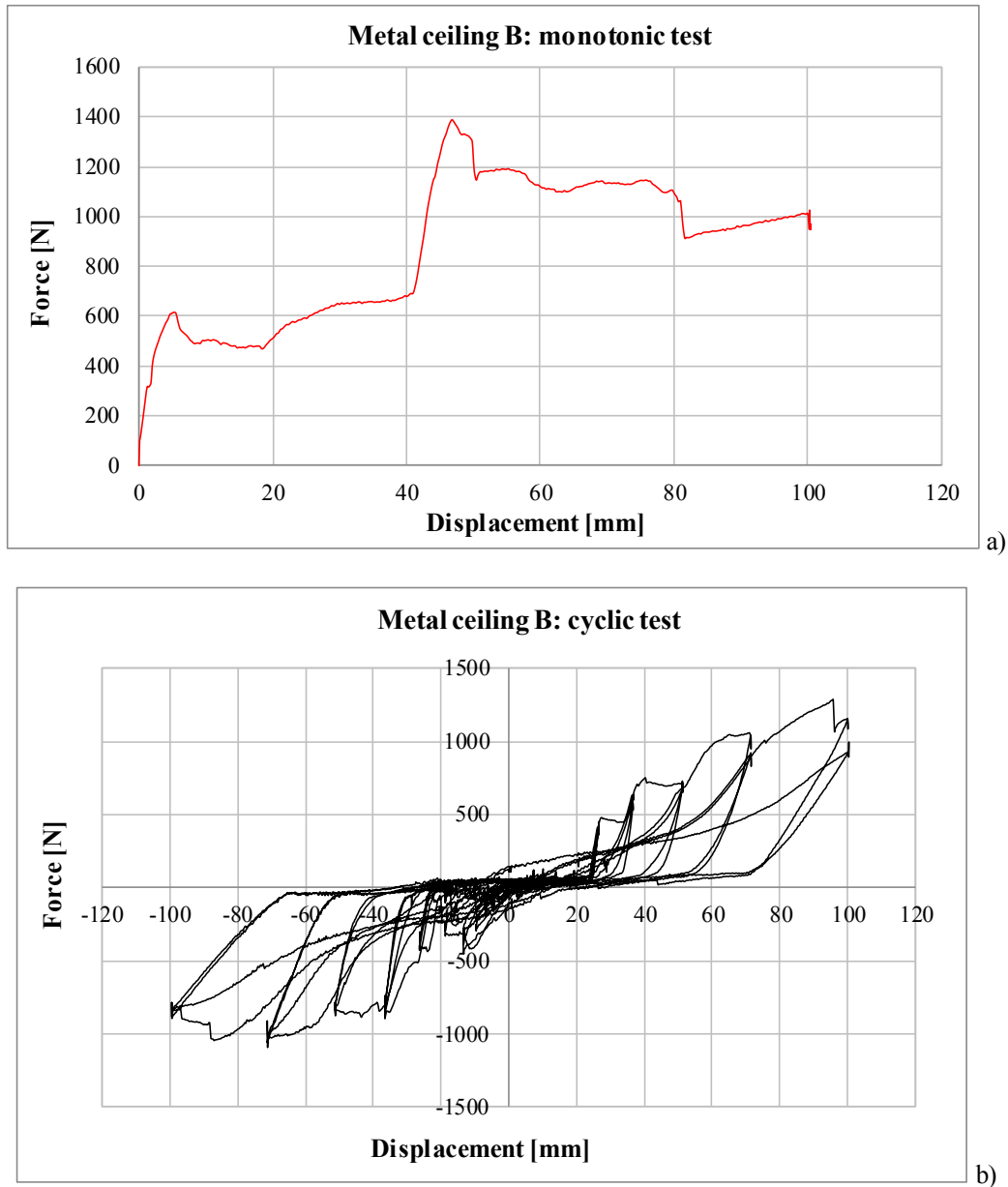


Fig. 60: Force-displacement graph of Metal ceiling B specimens: a) monotonic test, b) cyclic test.

The damage mechanisms that were observed are the same of the previous tests: buckling of the slotted studs of the seismic bracings, bending and deformation of the main runner connected to the seismic bracing. As illustrated in Fig. 44 c), in Metal ceiling B primary and secondary runners are connected with a bracket. During the tests the deformation on the primary runner caused damage in the bracket that induced the detachment of the secondary runner from the primary runner for a displacement of about 9 cm. During the monotonic tests, some tiles lower, following the damage on the secondary runner, but thanks to their particular edge shape they didn't collapse (Fig. 61 a). In fact, it can be noticed in Fig. 44 a) and b) that the edge of the tiles

is folded to follow the section of the secondary runner. In the cyclic test, this same phenomenon affecting the brackets determined the collapse of a tile for a displacement of 10 cm (Fig. 62 a). Other details of the damage during monotonic and cyclic tests are reported in Fig. 61 and Fig. 62.

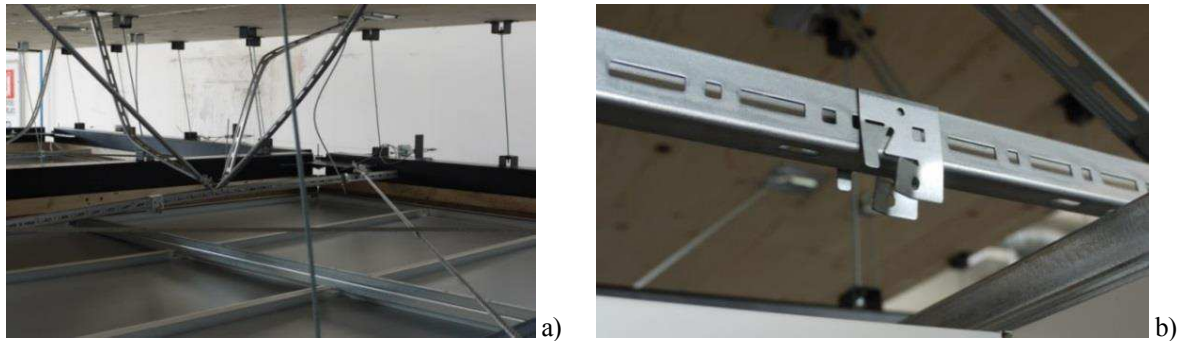


Fig. 61: Damage mechanisms of monotonic test on Metal ceiling B specimen: a) buckling of the slotted studs of the seismic bracing, b) rupture of the bracket connecting main and cross runner .

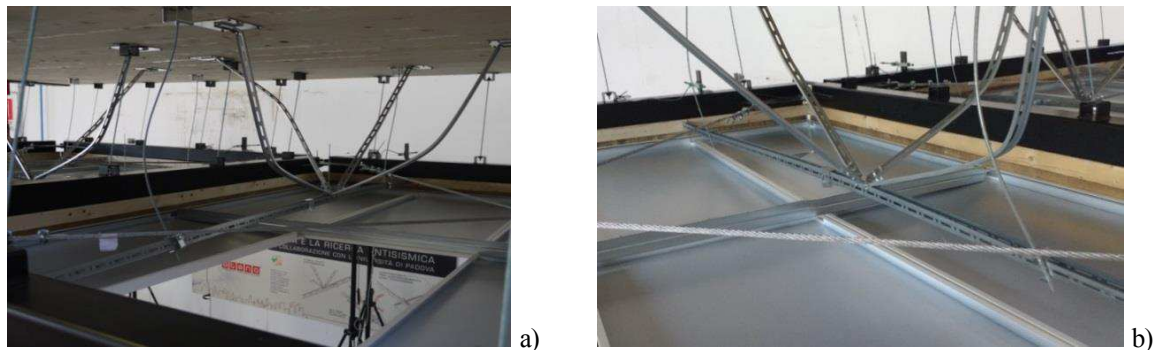


Fig. 62: Damage mechanisms of cyclic test on Metal ceiling B specimen: a) buckling of the slotted studs of the seismic bracing and collapse of a tile, b) rupture of the bracket connecting main and cross runner .

Since the results of the Metal ceiling B specimens showed high variability and high difference between monotonic and cyclic tests, the results were considered unreliable and are not analysed.

3.3.5 Plast Sing High

In Fig. 63, the results of the cyclic test on the specimen of Plast Sing High is reported, the force is relative to a single seismic bracing and the displacement is the mean displacement measured from potentiometers 1, 2, 3 and 7.

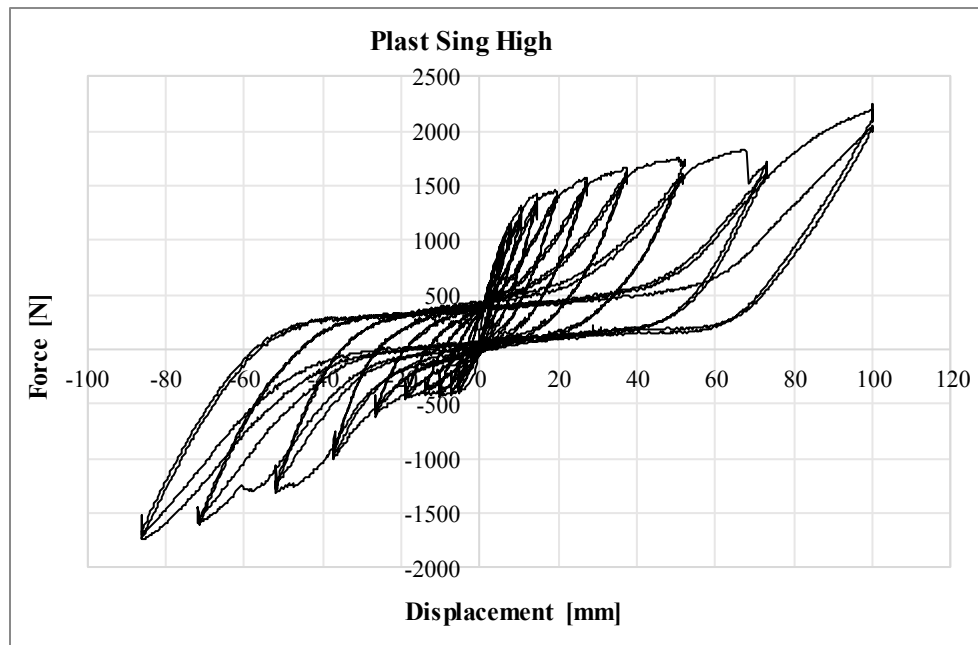


Fig. 63: Force-displacement graph of specimen Plast Sing High referred to mean value of displacements of potentiometer 1, 2, 3 and 7.

The graph indicated in Fig. 63, highlights an elasto-plastic behaviour with hardening. The specimen has an elastic behaviour until reaching 9 mm and the maximum resistances reached after the plastic branch is 2200 N and 1700 N for each direction and for each seismic bracing. As it can be noticed in Fig. 63, the behaviour of the specimen is different in the two directions of loading. In particular, in push loading the passage from elastic to plastic behaviour is clear and the gradient of the plastic branch is constant except for the last cycle. Otherwise, in the pull loading, the gradient of the plastic branch increases rapidly after 20 mm of displacement. Both seismic bracings showed buckling of the slotted studs under compression in the loading direction. The buckling of the studs caused the bending and uplift of the main runners where the seismic bracings were attached and consequently a compression force acting on the studs of the seismic bracings, placed orthogonally to the loading direction. The hardening behaviour that can be noticed during push loading is caused by the plasterboards that prevent the deformation of the main runner. As in the previous tests, during cyclic tests all the studs were under compression and in particular, the studs in the loading direction were alternatively under tension and compression forces, while the studs in the orthogonal direction were subjected only to compression forces. In the bracket connecting the seismic bracing to the main runner no bearing of the connection was observed and the only damage observed was the bending of runners in correspondence to the attachment of the seismic bracing (see Fig. 64 b). The plasterboards showed bending phenomena, due to the deformation of the system and the maximum deformation measured is 17 mm (see Fig. 64 c) and d). All the damage mechanisms are illustrated in Fig. 64.

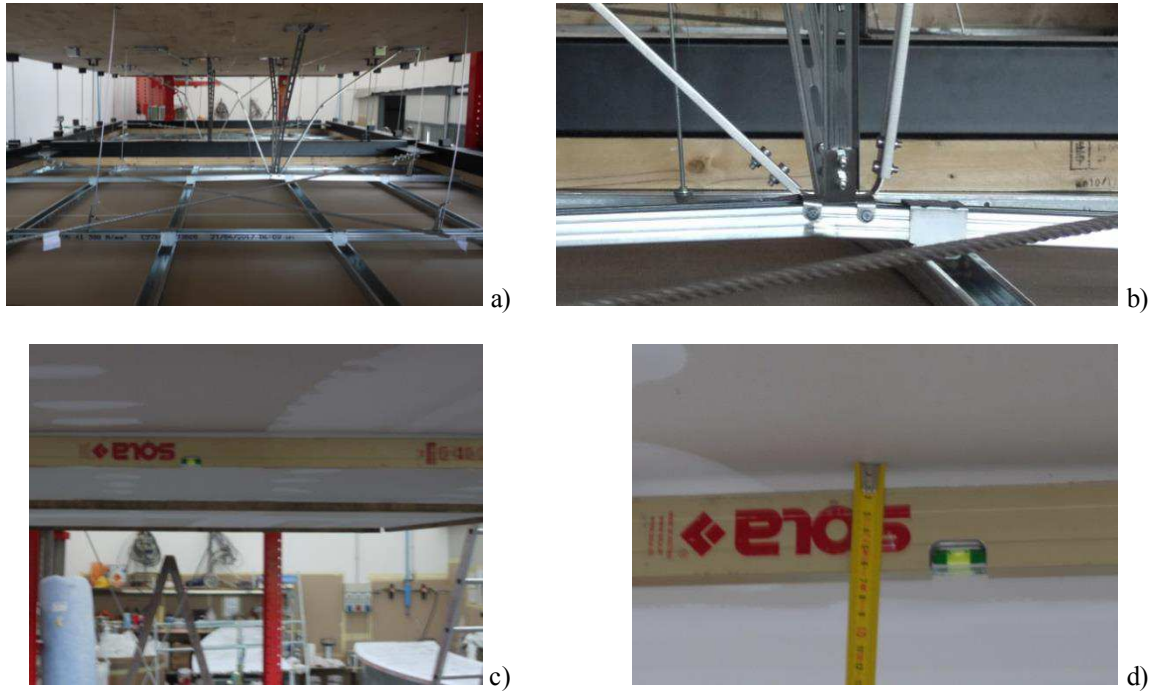


Fig. 64: Damage mechanisms of cyclic test on Plast Sing High specimen: a) buckling of the studs of the seismic bracing during cyclic test, b) detail of bending of the main runner in correspondence to the attachment of the main runner, c) and d) deformation of the plasterboard.

In Table 25, the value of force, yielding and the parameters calculated through the envelope curve and the bilinear function are reported (see Fig. 65). In Table 26, the values of viscous damping ratio are reported. It can be noticed that the values decrease with the increase of the cycles amplitude.

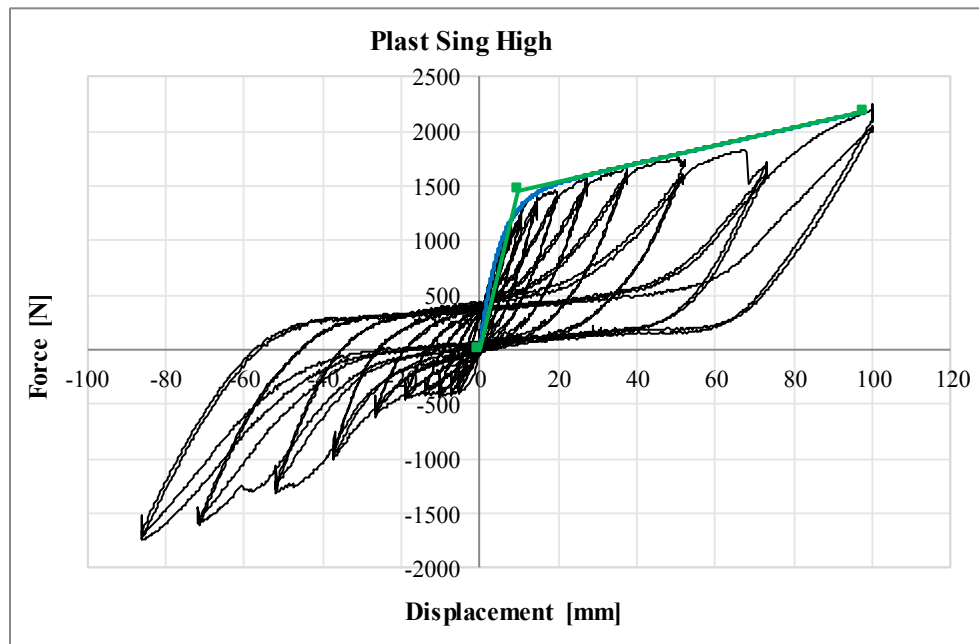


Fig. 65: Envelope curve and bilinear function for the specimen Plast Sing High.

Table 25: Analysis of the results from cyclic test on Plast Sing High specimen.

Parameters		Push loading
Maximum displacement	$\delta_{\max}$ (mm)	98.00
Ultimate force	F_{ult} (N)	2177
Maximum force	$F_{\max}$ (mm)	2177
Yielding displacement	δ_y (mm)	9.88
Yielding force	F_y (N)	1460.57
Ductility	$\mu = \delta_u / \delta_y$	79.92

Table 26: Viscous damping ratio of Plast Sing High specimen

Cycle amplitude (mm)	Viscous damping ratio v_{eq} (%)
5.47	10.74
7.17	9.04
10.01	9.25
13.82	9.28
19.11	7.95
26.64	7.61
36.92	7.43
50.65	8.11
72.00	6.53

3.3.6 Plast Sing Low

In Fig. 67, the results of the monotonic and cyclic tests on the specimens Plast Sing Low are reported. The only difference with Plast Sing High is the height of the void, that has changed from 0.67 m to 0.37 m. As consequence of the reduction of the void height, the uplift of the tubular beams was higher and the force acting on the hanger rods that hold the beams was also elevate. In order to avoid any damage to the setup, a different protocol was applied to the cyclic test (Fig. 66). The maximum displacement applied is reduced to 7 cm and the amplitude of the cycles is defined with the indication reported in FEMA 461 (2007).

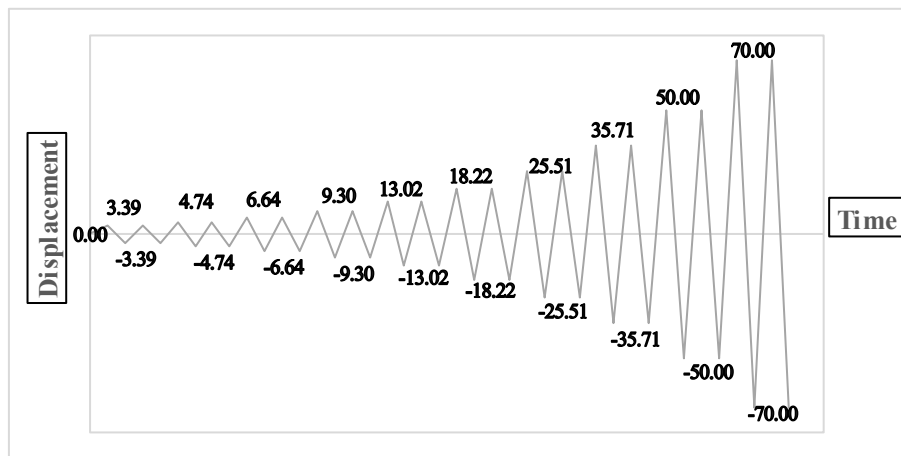


Fig. 66: Protocol for cyclic tests of specimens with void height equal to 0.37 m.

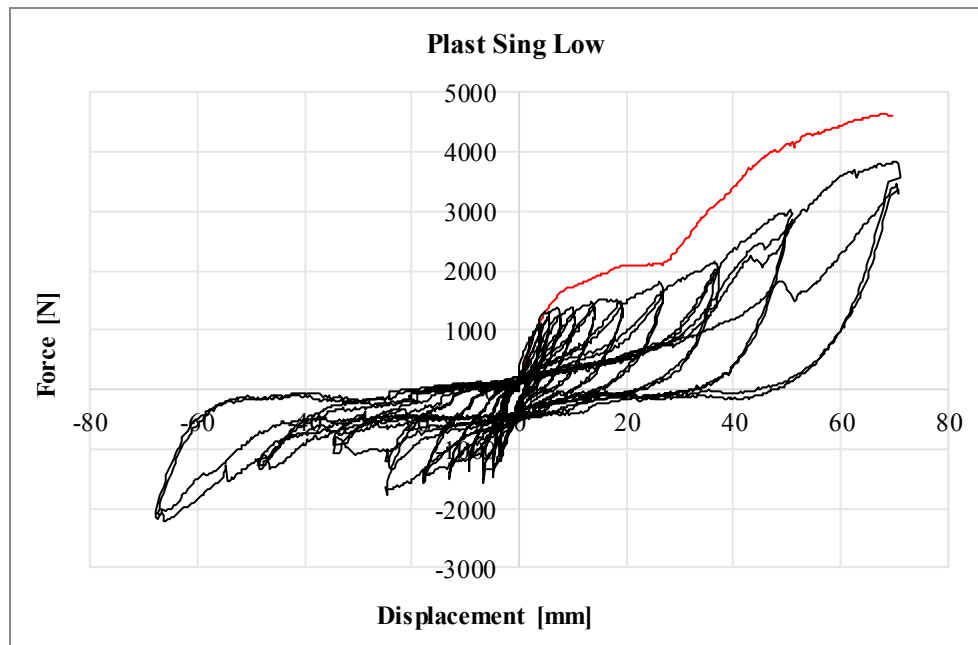


Fig. 67: Force-displacement graph of specimen 10 (in red) and specimen 11 (in black), referred to mean displacement of potentiometers.

The behaviour of Plast Sing Low is asymmetrical in push and pull loading and the maximum resistance reached is 3811 N in push and 2161 N in pull. This particular behaviour and the irregular results obtained in pull loading are caused by the high stiffness of the system and the small height of the void that determined in pull the distortion of the hanger rods of the tubular beams. The behaviour of the specimens in push is elasto-plastic with hardening.

As Plast Sing High, the damage mechanisms that affect the specimens of Plast Sing Low are buckling of the studs of the seismic bracing under compression (Fig. 68 a), the consequent uplift and bending of the main runner to which the seismic bracing is connected and the deformation of the web of the main runner where the seismic bracing is attached. The plasterboards and the brackets that connect the seismic bracing to the main runner weren't affected by any damage. In Fig. 68 and Fig. 69, the damage after monotonic and cyclic tests are reported.

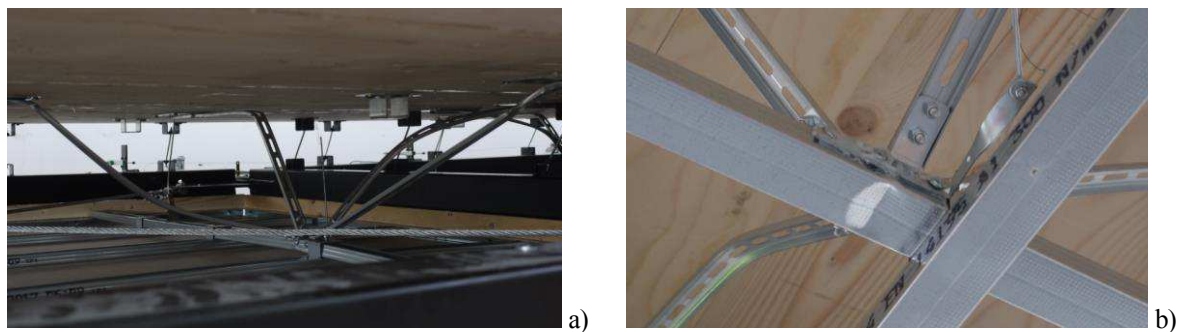


Fig. 68: Damage mechanisms of specimen Plast Sing Low after monotonic test: a) buckling of the studs of the seismic bracing, b) deformation of the web of the main runner.

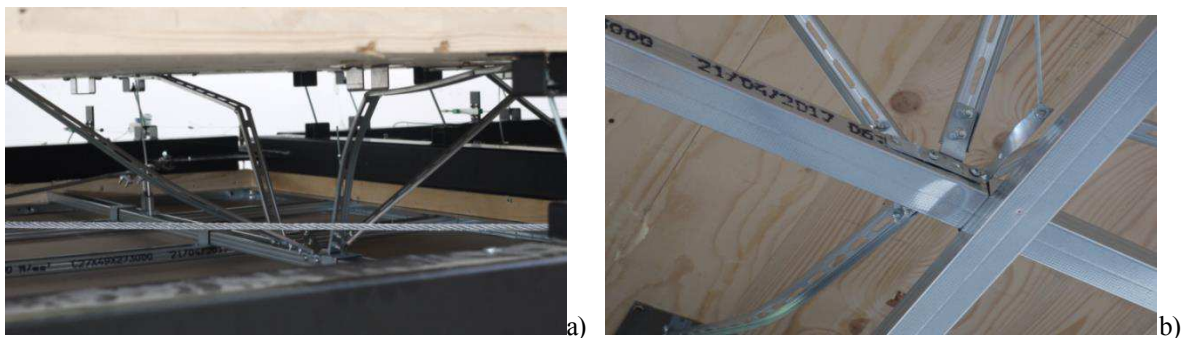


Fig. 69: Damage mechanisms of specimen Plast Sing Low after cyclic test: a) buckling of the studs of the seismic bracing in the left side c) deformation of the web of the main runner.

With the aim of realizing a reliable analysis of the experimental results, the maximum displacement considered for the envelope curve and the bilinear function (represented in Fig. 70) in push loading is 65 mm. In Table 27 the values of force, yielding and the parameters calculated through the definition of envelope and bilinear function are reported. In Table 28 the values of viscous damping ratio are indicated: with the exception of the first cycle they maintain a constant value for all cycles.

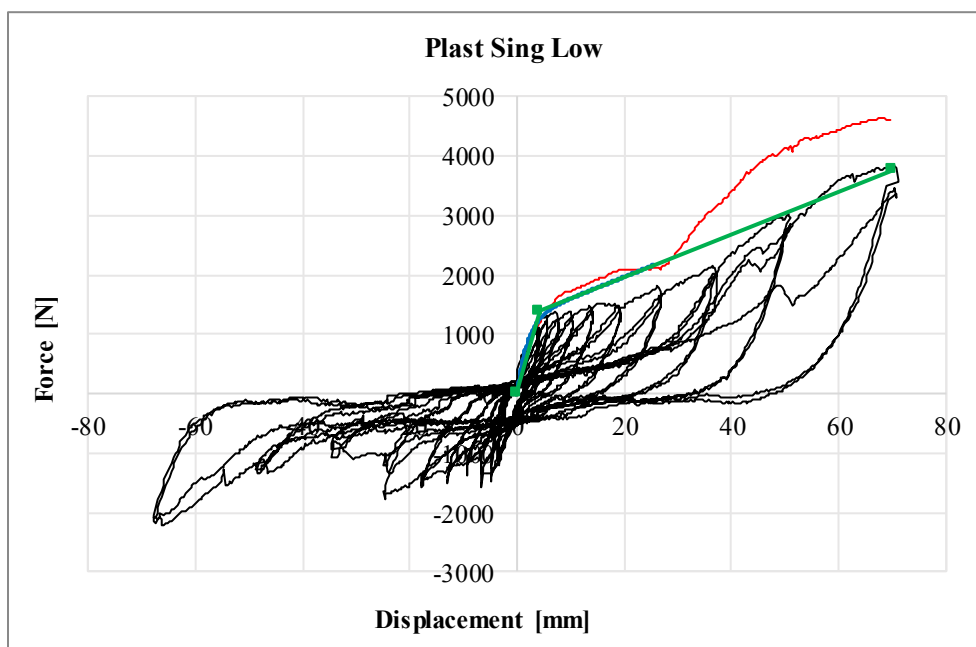


Fig. 70: Envelope curve and bilinear function for the specimen Plast Sing Low.

Table 27: Analysis of the results from cyclic test on Plast Sing Low specimen.

Parameters		Push loading
Maximum displacement	δ_{max} (mm)	70.00
Ultimate force	F_{ult} (N)	3770
Maximum force	F_{max} (mm)	3770
Yielding displacement	δ_y (mm)	4.22
Yielding force	F_y (N)	1401.80
Ductility	$\mu = \delta_u / \delta_y$	16.60

Table 28: Viscous damping ratio of the Plast Sing Low specimen.

Cycle amplitude (mm)	Viscous damping ratio v_{eq} (%)
4.02	18.40
5.65	14.07
7.77	12.68
10.46	13.13
14.19	14.12
19.31	14.00
27.02	13.56

3.3.7 Plast Doub Low

In Fig. 71 are represented the results of the Plast Doub Low specimen, the cyclic test is performed according to the protocol represented in Fig. 70. The value of displacement reported is the mean value of potentiometers 1, 2, 3 and 7.

Since the height of the void is low, the tubular beams have higher uplift due to the pendulum effect of the setup. The uplift of the beams couldn't be accommodated by the connector to the load cell and a concentration of force was developing in its connection area. In order to avoid any damage to the load cell, this phenomenon was limited executing one cycle with the maximum amplitude only in tension. The behaviour of the specimen Plast Doub Low is elasto-plastic with hardening and it reached a maximum resistance of 3459 N and 3943 N in the two directions.

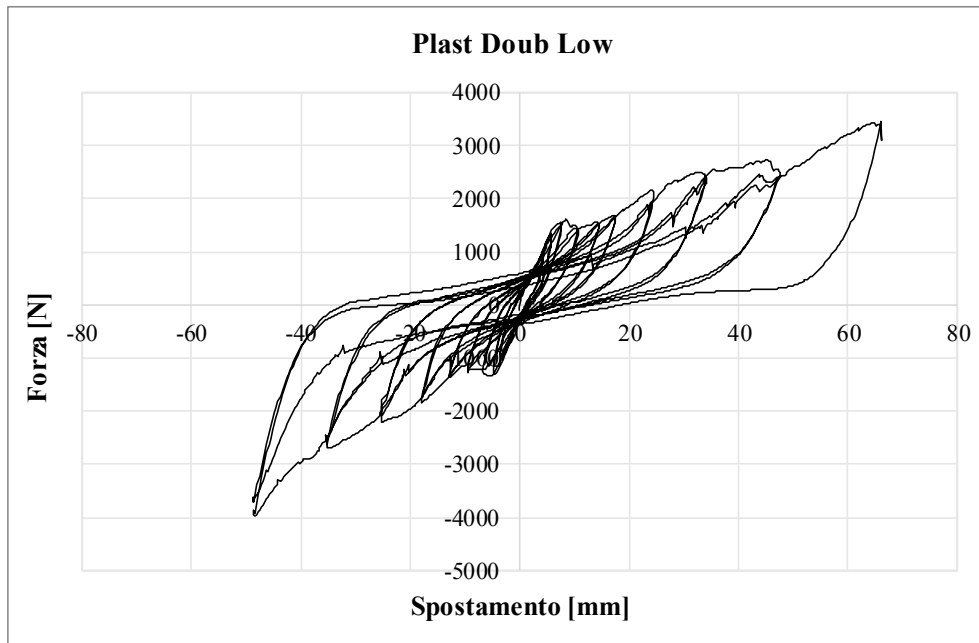


Fig. 71: Force-displacement graph of specimen Plast Doub Low

The damage mechanisms are the same of the previous specimens, e.g. buckling of the studs of the seismic bracing and uplift of the main runner to which it was connected (Fig. 72 a) and b). Moreover, for a displacement of 50 mm two hanger rods lose their connection to the main runner (Fig. 72 c) d) and e).

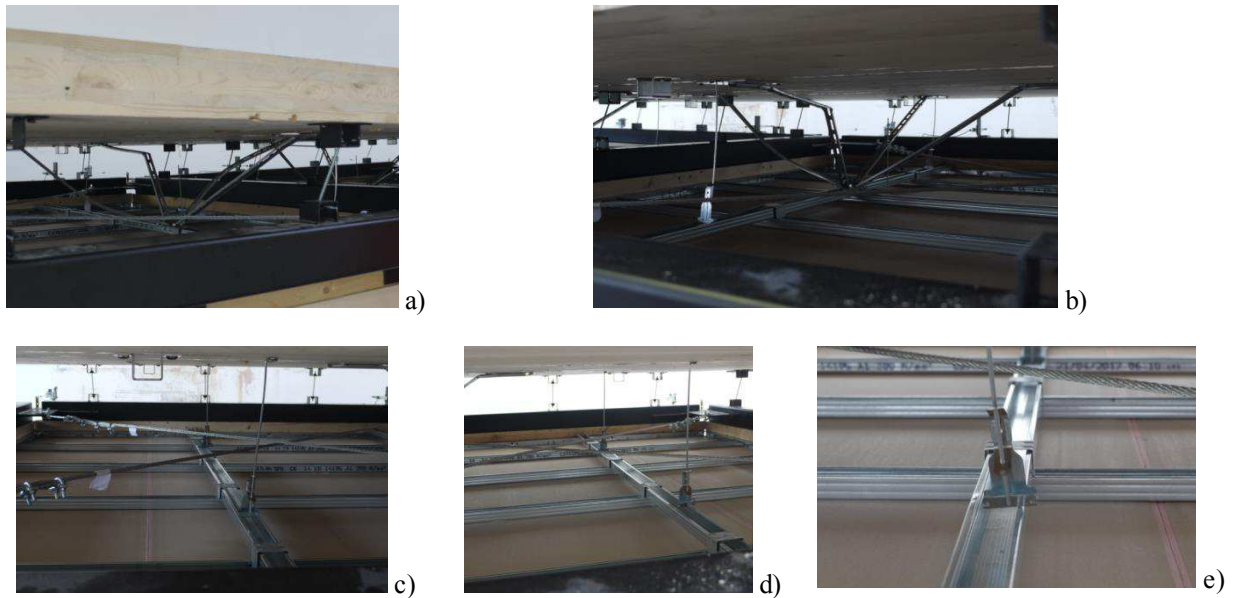


Fig. 72: Damage mechanisms of specimen Plast Doub Low: a) buckling of the studs of the seismic bracing in the left side, b) buckling of the studs of the seismic bracing in the right side c) detachment of the hanger rod in the left side d) detachment of the hanger rod on the right side e) particular of the detachment of the hanger rod.

As for the specimen of Plast Sing Low, the experimental results of Plast Doub Low were elaborated considering the push loading and a maximum displacement equal to 65 mm. The envelope curve and bilinear function are represented in Fig. 73. The parameters obtained after elaborating the results are indicated in Table 29. The values of viscous damping ratio are reported in Table 30: it is constant for almost all cycles but it increases for the last two cycles.

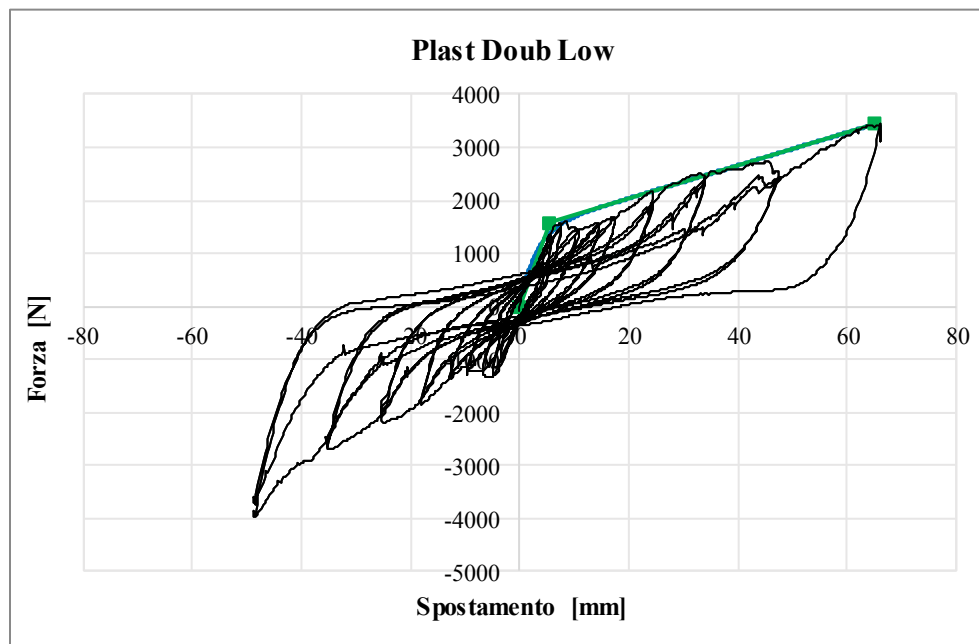


Fig. 73: Envelope curve and bilinear function for the specimen Plast Doub Low.

Table 29: Analysis of the results from cyclic test on Plast Doub Low specimen

Parameters		Push loading
Maximum displacement	$\delta_{\max}$ (mm)	65.00
Ultimate force	F_{ult} (N)	3432.00
Maximum force	$F_{\max}$ (mm)	3432.00
Yielding displacement	δ_y (mm)	5.48
Yielding force	F_y (N)	1570.83
Ductility	$\mu = \delta_u / \delta_y$	11.86

Table 30: Viscous damping ratio of the Plast Doub Low specimen.

Cycle amplitude (mm)	Viscous damping ratio v_{eq} (%)
4.32	11.57
5.74	10.60
7.62	12.20
10.74	13.70
14.46	12.72
17.42	12.99
24.34	11.73
34.01	21.35
47.68	24.10

3.4 Elastic stiffness and vibration period of the suspended ceilings

In this section, the elastic stiffness and vibration period of the ceilings studied are calculated. The elastic stiffness is calculated according to the procedure presented in the previous chapter and consequently all the typologies of specimens are studied with the exception of the Metal ceiling B.

The evaluation of the elastic stiffness of the bracing system allows to estimate the vibration period of the suspended ceiling with the formulation (2):

$$T_{el} = 2\pi \sqrt{\frac{M}{K}} \quad (2)$$

Where:

- M is the mass, in kg, pertinent to a single bracing system;
- K is the elastic stiffness evaluated experimentally.

The values of elastic stiffness and vibration period for each specimen analysed are reported in Table 31.

Table 31: Elastic stiffness and vibration period of specimens tested.

Typology of specimen	Loading	Elastic stiffness K_y [N/mm]	Vibration period [s]
T-Grid with metal tiles	Push	35.14	0.209
	Pull	50.06	0.175
T-Grid with lightweigh gypsum tiles	Push	59.8	0.160
	Pull	63.5	0.156
Metal ceiling A	Push	258	0.077
	Pull	259	0.077
Plast Sing High	Push	147.8	0.120
Plast Sing Low	Push	332.4	0.080
Plast Doub Low	Push	286.8	0.086

The T-Grid typology with metal tiles is the system with the lowest elastic stiffness, followed by the same system with lightweight gypsum tiles. They are characterized for being the lightest ceilings with the highest void height. Otherwise, Metal ceiling A has a high value of elastic stiffness, in particular if compared to the Plast Sing High specimen, that has similar void height but higher weight. From literature, the drywall suspended ceilings were expected to have a more rigid behaviour than metal ceilings. This consideration isn't confirmed by the data here presented, as the Plast Sing High specimen has an elastic stiffness value lower than Metal ceiling A. The Plast Sing Low specimen has a high value of elastic stiffness, in fact it is higher than the elastic stiffness of Plast Sing High (which has a lower void high) but it is also higher than the elastic stiffness of Plast Doub Low, whose difference is only in the higher weight. Finally, the conclusion is that there is a insufficient number of tests results to define a relation between elastic stiffness, typology of specimen and void height.

3.5 Evaluation of the buckling load

Since the main damage mechanism of all suspended ceilings is the buckling of the bracing system studs, in this section the critical buckling force will be evaluated analytically as function of the void height for each test.

The analytical total resistance of the bracing system is calculated as the sum of the forces of two parallel mechanisms (Fig. 74). In the first mechanism, only the studs installed in the direction parallel to loading are considered (mechanism 1: stud 1 under tension, stud 4 under compression). Otherwise, the second configuration considers the studs positioned orthogonally to the loading direction and the studs under tension in the loading direction (mechanism 2: stud 1 under tension, studs 2 and 3 under compression).

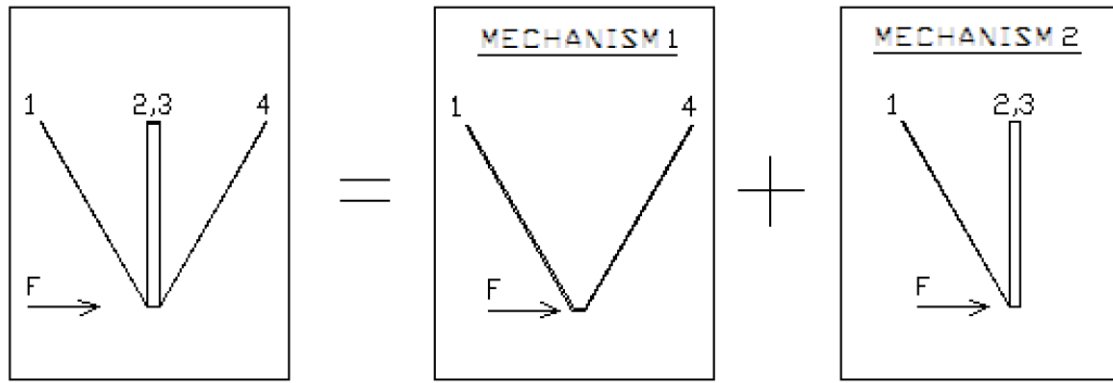


Fig. 74: Parallel mechanisms for the evaluation of the buckling analytical resistance of the bracing system.

For each mechanism the buckling critical loads are evaluated and the resultant forces are determined as sum of their projection in the horizontal loading direction. With this procedure, the analytical and experimental values of resistance of the bracing systems can be compared. The hypotheses for these evaluations are that the weight of the ceiling is totally supported by the hanger rods and the bracing systems alone carry the lateral force.

The critical load for a real stud was evaluated according to NTC08, § 4.2.4.1.3. In particular, the effective length (L_0 in Fig. 75) was evaluated for each suspended ceiling and void height. The specimens 1, 2, 3 and 4 have the studs inclined at 60° to the ceiling plane, the specimens 5, 6, 7, 8 and 9 have 45° -inclination and the specimens 11 and 12 have 30° -inclination. In Table 32, the length of each stud is indicated.

To compute the real critical load, the Euler's critical load was calculated according to the formulation (3):

$$N_{cr} = \pi^2 \frac{EJ}{l_0^2} \quad (3)$$

Where:

- E is the elastic modulus of the bracing stud, assumed equal to 210000 MPa;
- J is the minimum moment of inertia of the C-section of the bracing stud;
- $l_0 = \beta \cdot l$ is the effective length of the stud under compression.

The type of connections of the bracing studs to the main runner and the soffit are partial fixed joints whose rotational stiffness is not known and cannot be defined accurately experimentally. Consequently, it wasn't easy to determine the reduction factor β to obtain the correct effective length of the studs and calculate the value of critical buckling load. Then, the process was inverted and the factor β was chosen in order to have the best correspondence between analytical and experimental results. In Table 32, the effective length used and the reduction factors are indicated. Moreover, in Table 33, the evaluation of the theoretical buckling critical load is reported for each system tested. In Table 34, the values of horizontal resistance are indicated. They are calculated superimposing the forces obtained in the configurations previously described.

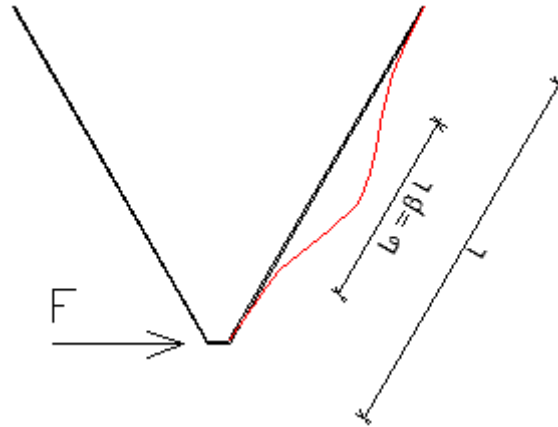


Fig. 75: Representation of the effective length of the bracing system studs.

Table 32: Length of the bracing studs.

Typology of specimen	L [mm]	β	L_0 [mm]
T-Grid –metal tiles	1290	0.65	838
T-Grid - lightweight gypsum tiles	1290	0.65	838
Metal ceiling A	885	0.6	531
Metal ceiling B	856	0.8	684
Plast Sing High	871	0.6	523
Plast Sing Low	632	1	632
Plast Doub Low	632	0.75	474

Table 33: Real critical buckling load.

Typology of specimen	Euler's critical load N_{cr} [N]	Critical buckling load for real stud $N_{b,Rd}$ [N]
T-Grid metal and lightweight gypsum tiles	505	427
Metal ceiling A	1257	990
Metal ceiling B	757	623
Plast Sing High	1007	810
Plast Sing Low	689	571
Plast Doub Low	1225	967

Table 34: Evaluation of horizontal force for real stud – variable L_0 .

Typology of specimen	Force mechanism 1 [N]	Force mechanism 2 [N]			Total horizontal force: mechanism 1 + mechanism 2 [N]
		$F_{\text{vert},2x}$	$F_{\text{stud_tens}}$	$F_{\text{horizz_stud_tens}}$	
T-Grid metal and lightweight gypsum tiles	213	739.5	854	427	640
Metal ceiling A	700	1400	1980	1400	2101
Metal ceiling B	441	882	1247	882	1323
Plast Sing High	573	1146.2	1621	1146	1719
Plast Sing Low	495	571	1143	990	1484
Plast Doub Low	838	967.5	1935	1676	2514

The comparison with the experimental results is indicated in Table 35. The maximum experimental strength was evaluated as the mean value of the maximum forces reported for each typology tested.

Table 35: Comparison analytical/experimental resistance for real stud - variable L_0 .

System	Analytical load [N]	Experimental load [N]	Analytical/Experimental ratio
T-Grid - metal and lightweight gypsum	640	604	1.06
Metal ceiling A	2101	2058	1.02
Metal ceiling B	1323	1323	1.00
Plast Sing High	1719	1653	1.04
Plast Sing Low	1484	1524	0.97
Plast Doub Low	2514	2405	1.05

As indicated in Table 32, for each typology of specimen a different value of β is chosen. It can be related on many characteristics e.g. the void height, the characteristics of the bracing studs and their connection to the runner and the soffit or the direction of loading in relation to the direction of the main tees. However, the restricted number and combination of tests don't allow to establish a relation between these characteristics and the value of β . Consequently, the suggestion is to use $\beta=1$ in the design practice and so, to consider the effective length equal to the unrestrained length of the stud. This choice is also confirmed by the deformed configuration evaluated experimentally.

The evaluation of the analytical resistance of the bracing system is reported in Table 36, with the case $\beta=1$.

Table 36: Real critical buckling load.

Typology of specimen	Euler's critical load N_{cr} [N]	Critical buckling load for real stud $N_{b,Rd}$ [N]
T-Grid metal and lightweight gypsum tiles	213	188
Metal ceiling A	453	385
Metal ceiling B	485	411
Plast Sing High	363	312
Plast Sing Low / Plast Doub Low	689	571

Table 37: Evaluation of horizontal force for real stud – $L_0 = 1$.

Typology of specimen	Force mechanism 1 [N]	Force mechanism 2 [N]			Total horizontal force: mechanism 1 +mechanism 2 [N]
		$F_{vert,2x}$	F_{stud_tens}	$F_{horizz_stud_tens}$	
T-Grid metal and lightweight gypsum tiles	94.0	325.5	376	188	282
Metal ceiling A	272	545	771	545	817
Metal ceiling B	291	581	822	581	872
Plast Sing High	221	442	625	442	663
Plast Sing Low / Plast Doub Low	495	571	1143	990	1484

3.6 Conclusions

The experimental setup realized has the aim to test the behaviour of suspended ceilings subjected to increasing loading cycles in their plane. Different typologies of suspended ceilings were tested and for each one, one monotonic and one cyclic test were performed. The specimens tested are different for void height and manufacturing. In fact, T-Grid suspended ceilings with metal and lightweight gypsum tiles are considered, other than two more different metal ceilings. Moreover, two drywall suspended ceilings are tested, with single and double plasterboard. All the specimens have in common the hanger rods and seismic bracing systems, whose configurations are similar and differ only for the attachment details to the runners.

The results of the tests allow to evaluate the seismic performance of the ceilings in terms of resistance and behaviour of the seismic restraints (bracing systems), that is assumed to resist to lateral loading. Furthermore, any eventual damage to the other components of the ceilings is evaluated.

The preliminary study of the bracing system outlined two possible failure mechanisms: failure of the connection at the base between main runner and the bracket of the bracing system and buckling of the studs. The experimental tests reported only buckling of the studs while the

connections of the bracing system evidenced just local yielding and embedment without failure. In fact, the buckling phenomenon is the main damage observed and it affected all the specimens, in both monotonic and cyclic tests. While in monotonic tests the buckling was observed in three compressed studs (that in the loading direction and the two in the orthogonal direction), in cyclic tests the buckling affected all the four studs alternatively. The buckling phenomenon caused the damage of the main runner to which the seismic bracing was attached. The specimens reacted differently according to their construction details. T-Grid has a good behaviour, particularly when metal tiles were installed, as they could accommodate the deformation induced by the main runners. T-Grid with lightweight gypsum tiles showed a partial collapse of a tile, in fact they are more rigid than the metal tiles and couldn't deform accordingly. These two typologies are representative of the widespread T-Grid categories of suspended ceilings and the tests confirmed their capacity to deform in consequence of the cycling loading. Otherwise, the other metal ceilings showed partial or total collapse of one tile in consequence of the damage on the elements that support the tiles (Metal ceiling A) or that connect the primary to the secondary runners (Metal ceiling B). In these samples the weakest points of the suspended ceilings were in the connection elements between their many components. The Metal ceiling B specimens have bigger tiles than the other specimens but there aren't enough data to consider the dimension of tiles a factor influencing the resistance or the damage mechanism. The behaviour of the drywall suspended ceilings is mainly governed by their assembly characteristics. In fact, the characteristic of continuity of their plasterboards and their connection with screws to the secondary runners make them a more rigid structure whose behaviour can be considered as a whole. The bending deformation of the plasterboards was the consequence of the deformation induced by the main runner. Only in the specimen with double plasterboard the loss of connection of some hanger rods was reported without global failure of the false ceilings. Therefore the global behaviour of these typologies can be considered as good, despite of their intrinsic fragility.

The results obtained were elaborated and they permitted to calculate some mechanical parameters useful to identify the behaviour and dissipative capacity of bracing systems. As the buckling is the observed main failure mechanism for bracing, the critical buckling load was calculated according to the deformation shown during the tests and considerations on how to design the bracing systems have been reported.

Finally, the key parameters that affect the behaviour of the specimens and the resistance of the seismic bracings are:

- the void height;
- the inclination of the studs of the bracing system;
- the connection of the bracing to the main runner;
- the weight of the specimens;
- the configuration of the specimens;

To deeply investigate these factors other experimental campaigns need to be developed in order to have a sufficient number of specimens covering all the possible combinations of typologies, geometries and loading conditions.

4 Seismic demand and capacity assessment of suspended ceilings

In this chapter the experimental results that have been processed will be combined with the results from linear time history analyses of multi-story buildings to study the behaviour of suspended ceilings installed at various levels.

In detail, the aim is to define a “*displacement-based*” procedure for the design of suspended ceilings. The seismic demand on suspended ceilings, in terms of displacement and acceleration, is investigated with a different methodology from the “*force-based*” approach proposed by the NTC08 (2008) and EC8 (CEN 2013) for computing the seismic force on non-structural components.

Linear Time-History analyses were performed on five frame structures with different vibration periods in order to assess the floor seismic demand and to define the filtering action of the main structure on the non-structural element installed at different levels. The elastic floor spectra and the capacity curves are defined in an ADRS (Acceleration Displacement Response Spectrum) domain. The seismic demand of suspended ceilings in terms of acceleration and displacement is compared to their capacities. Finally, the results from the analyses are compared with the forces computed with the formulation proposed by the NTC08.

4.1 Numerical simulations

4.1.1 Numerical model

Five regular frames, with six storeys and different periods of vibration, are modelled with the OpenSees software (McKenna et al., 2000) to evaluate the seismic demand on the suspended ceilings. The periods were chosen to be consistent with five real cases and in order to evaluate the filtering action that the structure induces on the non-structural element. Table 38 shows the periods assumed for each case study.

Table 38: Vibration periods of steel frames.

Case study	N. Storey	1° mode period [s]	2° mode period [s]	3° mode period [s]
Structure 1	6	0.30	0.10	0.06
Structure 2	6	0.60	0.20	0.12
Structure 3	6	0.90	0.31	0.19
Structure 4	6	1.40	0.48	0.30
Structure 5	6	2.00	0.70	0.44

The frames have a total height of 22.2 m and interstorey height equal to 3.7 m. Columns are assumed pendular and the beams are considered infinitely rigid. The stiffness of the vertical bracing system was modified in order to obtain the desired 1st mode vibration period. The interstory mass is 28.2 t for each floor except for the last floor, where it is 18.3 t.

According to the *Floor Response Spectrum Method* (already discussed in Chapter 1.4), the numerical analyses were performed on the building without considering the suspended ceiling systems.

Linear Time-History (LTH) analyses were performed and each structure has been subjected to 7 accelerograms spectrum-compatible with the SLD elastic design for a type C soil in the high-seismicity territory of Amatrice (Rieti - Italy). Table 39 shows the parameters used for the definition of the design spectrum (Fig. 76) according to the guidelines provided in NTC08 (2008) (Table C7.1.I). The results from the analyses were then used to evaluate the seismic response of the suspended ceilings.

Table 39: Parameters used for the definition of the target spectrum (SLD).

Location	Limit state	Soil category	a_g [g]	S [-]
Amatrice	SLD	C	0.103	1.5

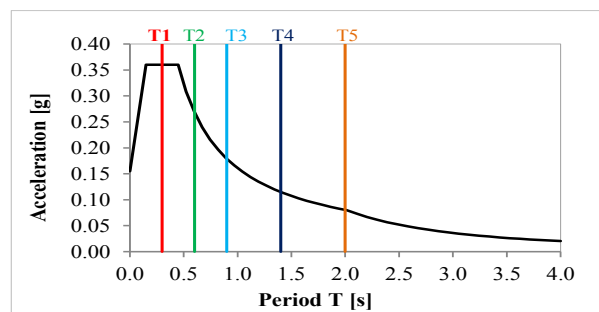


Fig. 76: Elastic design spectrum (SLD – Amatrice)

4.1.2 Floor response spectra

The software SeismoSignal (Seismosoft, 2002) was used to define the elastic acceleration and displacement floor response spectra at each level of each frame. A simple oscillator (SDOF system) with variable period was subjected to the floor accelerations evaluated with the LTH analyses assuming 5% damping. The floor response spectra were then evaluated using the response peaks of the oscillator.

4.1.2.1 Acceleration floor response spectra

Fig. 77 shows the mean values of the acceleration elastic response spectra (FRS) resulting from data processing.

The graphs of each structure have the same pattern and are characterized by spectral acceleration peaks at the fundamental periods of the building.

The filtering action of the structure causes the increase of seismic floor acceleration with the height of the floor (Fig. 77). In particular, Fig. 77 b) ($T=0.6s$) shows that at the fundamental period of vibration the first floor is affected by an acceleration of 0.78g while the sixth has 4.1g, meaning 5.25 times larger. As the building period increases, the peak spectral acceleration values decrease: in Fig. 77 a) ($T=0.3s$) the maximum accelerations at the last and at the first floor for the third vibration period of the building, are respectively 4.93g and 1.73g; while in Fig. 77 e) ($T=2s$) the same values are equal to 0.85g and 1.61g.

4.1.2.2 Displacement response spectra

Double integration of the SDOF equations of motion used for the definition of the acceleration FRS was performed in order to obtain the displacement floor response spectra (Fig. 78). These spectra exhibit a pattern characterized by peaks of displacement at the fundamental periods of the building.

As the period increases, the deformability of the building and the displacement demand increase. Similarly to acceleration floor response spectra, the increase of the displacement demand is caused by the higher quota of installation of the non-structural element.

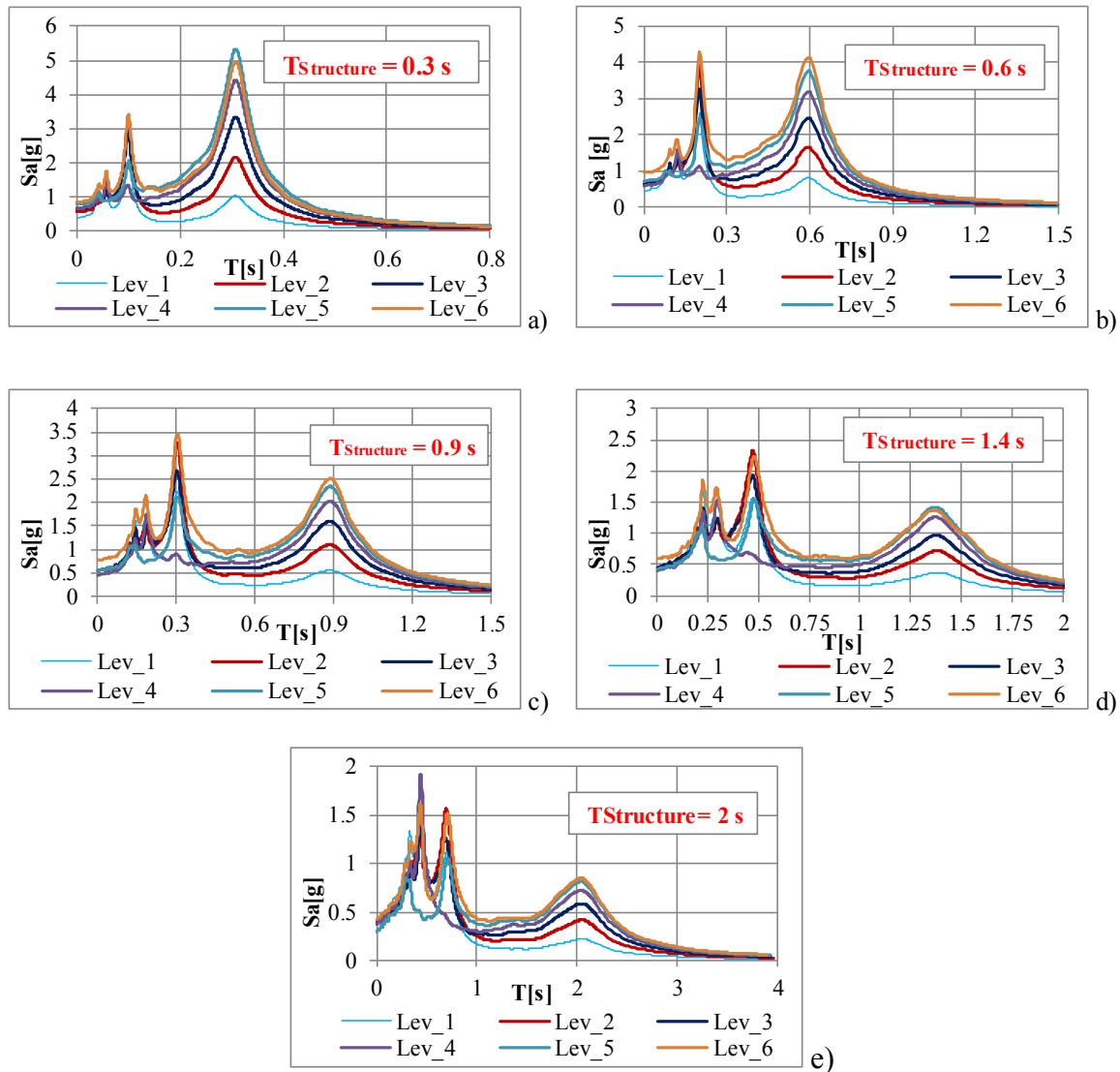


Fig. 77: Acceleration floor response spectra at each level for each case study (damping $\xi=5\%$).

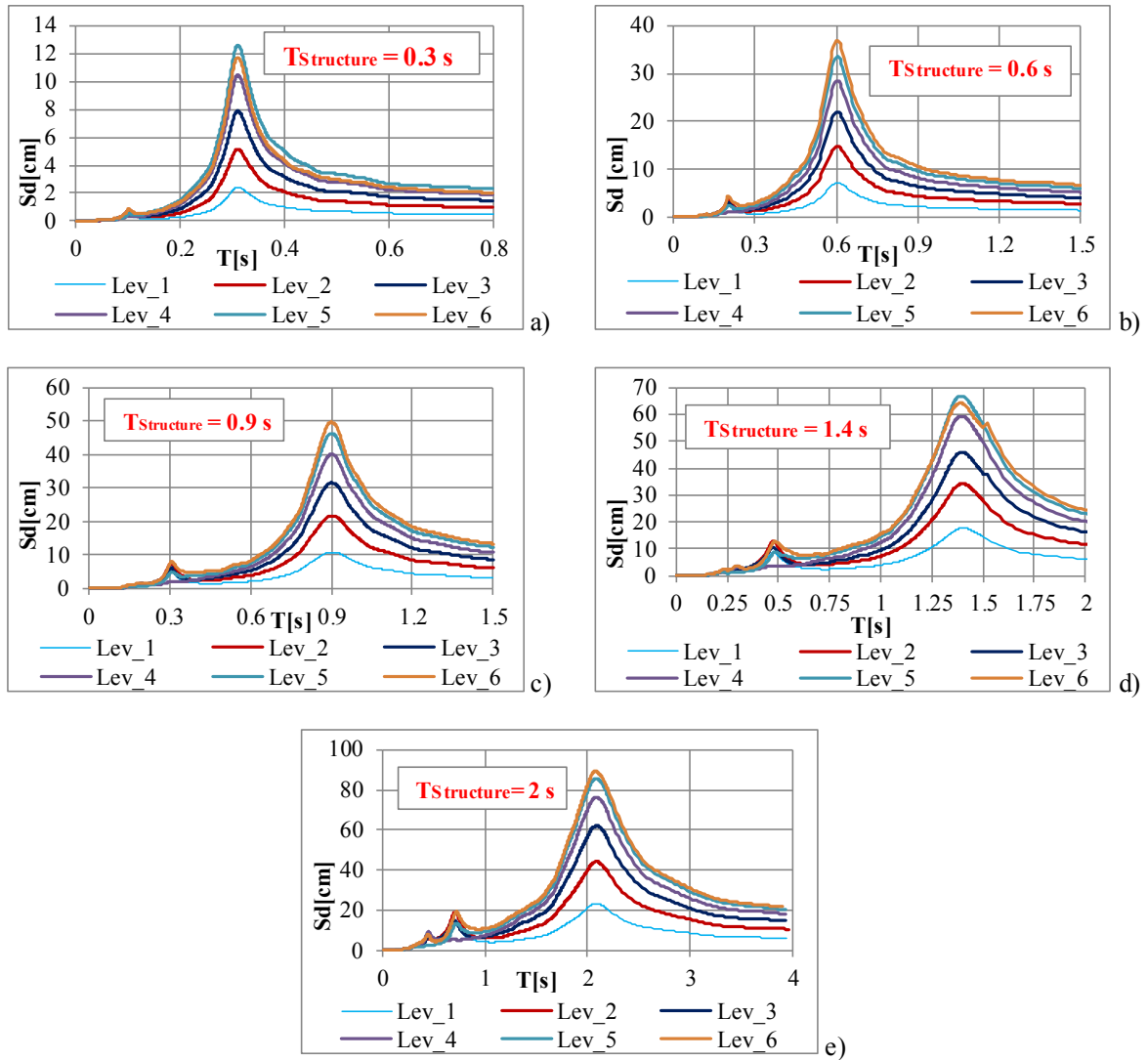
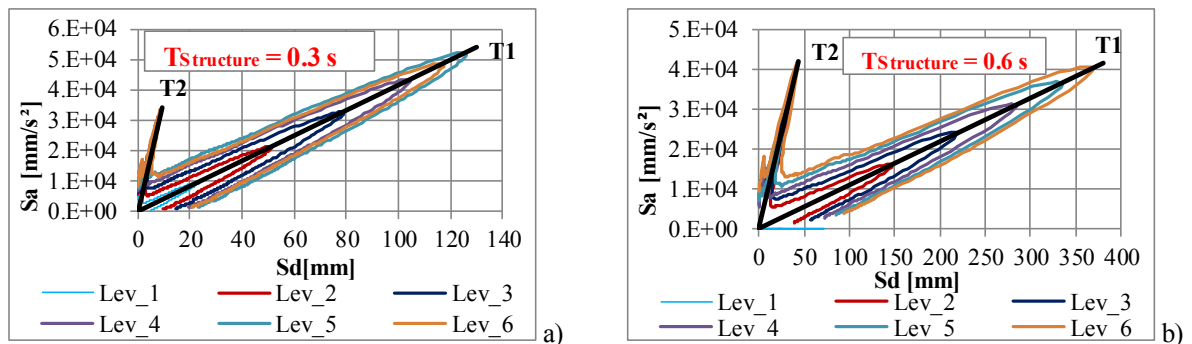


Fig. 78: Displacement floor response spectra at each level for each case study (damping $\xi=5\%$).

4.1.2.3 ADRS floor response spectra

The synthesis in a unique graph of the acceleration and displacement spectral coordinates defines the floor response spectra in the Acceleration Displacement Response Spectrum (ADRS) domain (Fig. 79). They are characterized by peaks at the fundamental periods of the main structure. The vibration periods of the structure are graphically defined by the gradient of the straight lines starting at the origin of the graphs. A lower gradient defines a higher vibration period.



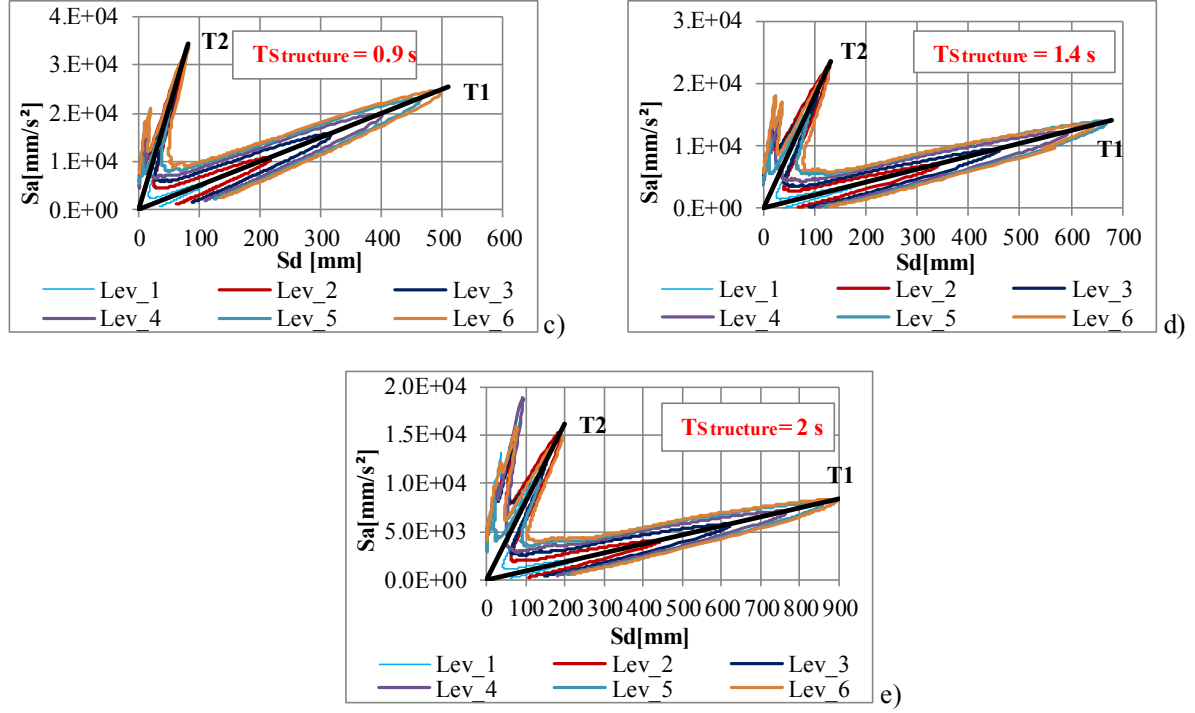


Fig. 79: ADRS floor response spectra for each case study (damping $\xi=5\%$).

4.2 Acceleration and displacement seismic demand assessment

The seismic demand on the specimen was evaluated with the experimental data of the specimen T-Grid with Metal Tiles, whose details are described in Chapter 3.2.1.1.

The total area of the system is 11.52 m² and seismic mass corresponding to a single seismic bracing system is 39 kg, the void height is 1.15 m.

The seismic demand of the suspended ceilings was evaluated processing the envelope curve and bilinear function of the experimental results, presented in Fig. 52.

A coordinate transformation was performed to convert the bilinear curve from force-displacement domain to spectral domain and to define the Capacity Spectrum (CS). The equations for the transformation are that used to define the capacity curves in pushover analysis of structures:

$$S_a = \frac{F}{\alpha_1 M} \quad (3)$$

$$S_d = \frac{\Delta}{\Gamma_1 \phi_{1r}} \quad (4)$$

$$\alpha_1 = \Gamma_1 \frac{\phi_1^T M I}{M} \quad (5)$$

$$\Gamma_1 = \frac{\phi_1^T M I}{\phi_1^T M \phi_1} \quad (6)$$

Where:

- F is the experimental strength;
- M is the seismic mass of the suspended ceiling;
- α_1 is the modal mass coefficient of the 1st mode defined in (5);
- Δ is the experimental displacement;
- Γ_1 is the participation factor of the 1st mode defined by (6);
- ϕ_1^T is the vector of the 1st modal form.

The suspended ceiling is considered as a SDOF system characterized by a single mode of vibration ($\alpha_1=1$; $\phi_1=1$; $\Gamma_1=1$).

Therefore, the previous equations are simplified in:

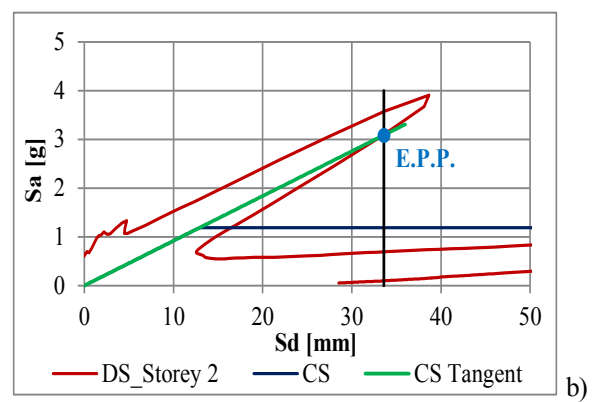
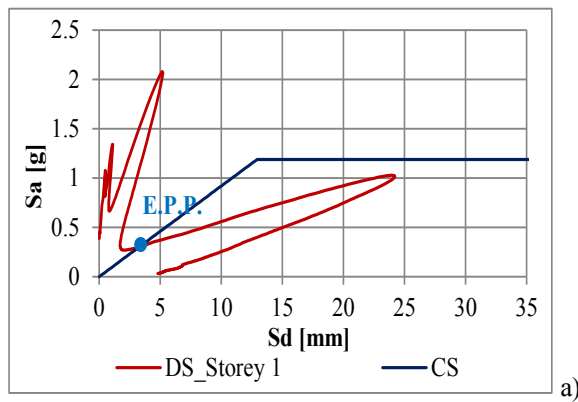
$$S_a = \frac{F}{M} \quad (7)$$

$$S_d = \Delta \quad (8)$$

Assuming an elastic behavior of the suspended ceiling, the target displacement and acceleration points have been defined (Elastic Performance Point - EPP). The intersection of the Demand Spectrum (DS) with the tangent to the origin of the CS defines the EPP for each floor of the five structures. Fig. 80 reports some specific cases and Fig. 81 summarizes the 30 values of elastic displacements obtained (6 storeys per 5 structures). The maximum value obtained is 40 mm at the last floor of the building with period 0.6 s.

The trend of the graphs is characterized by peaks of displacement when the period of the non-structural element is close to the period of the structure (Fig. 80 b).

Suspended ceilings installed in buildings with high vibration period ($T=1.4$ s– 2 s) have constant displacement values as the floor height increases. This trend is typical of systems with a period distant from the fundamental period of the structure. In these conditions suspended ceilings are not affected by resonance phenomena.



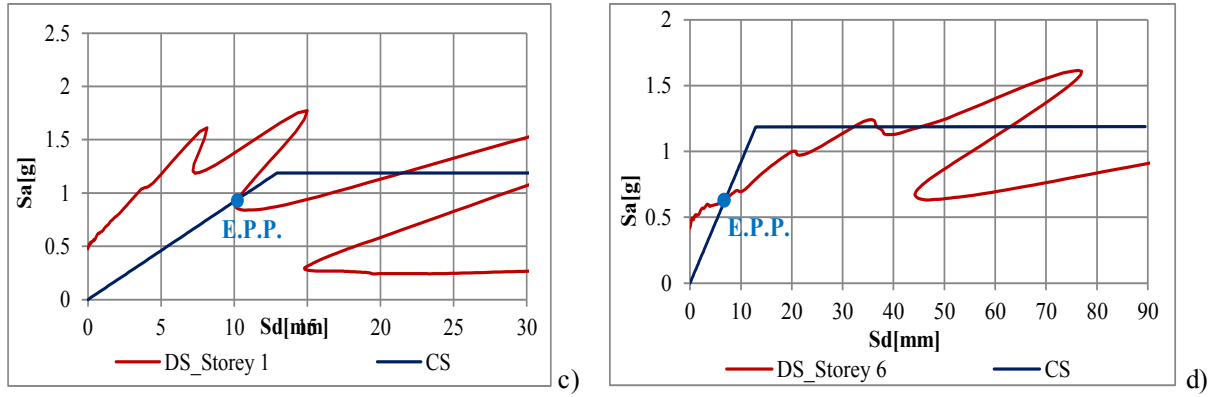


Fig. 80: Definition of EPPs in ADRS domain: a) Floor 1, structure $T=0.3s$, b) Floor 2, structure $T=0.6s$, c) Floor 1, structure $T=0.9s$, d) Floor 6, structure $T=2s$.

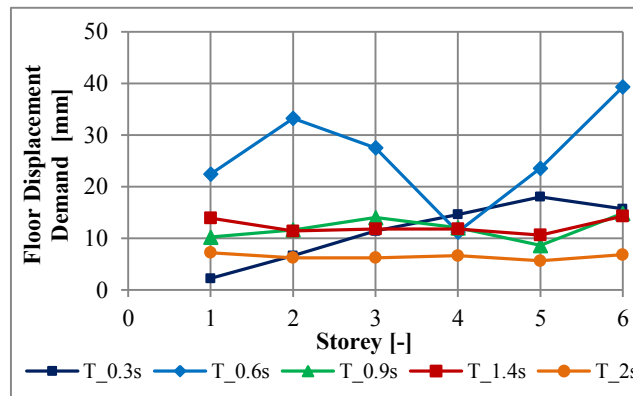


Fig. 81: Elastic displacement demand of suspended ceiling for each case study.

4.3 Evaluation of seismic force: numerical versus analytical values

The anelastic acceleration value acting on the suspended ceiling was determined by projecting the EPP on the bilinear curve (Fig. 82) and obtaining the anelastic performance points (APP). The hypothesis of equal displacements in elastic and anelastic behaviour of the suspended ceiling was assumed in order to evaluate the APP (Albanesi 2007). However, this hypothesis need to be confirmed with the execution of additional nonlinear numerical analyses.

Fig. 83 shows the comparison between elastic and anelastic PP in terms of acceleration and force.

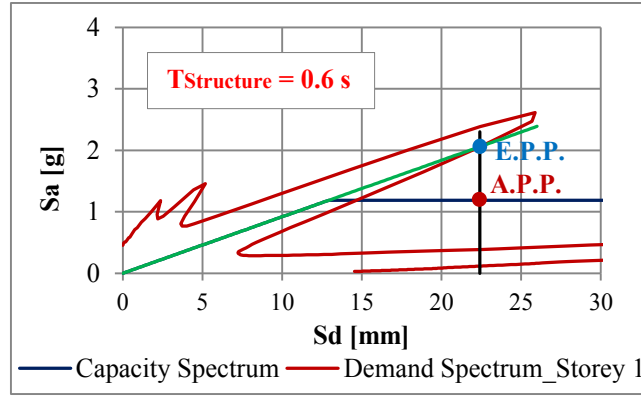


Fig. 82: Definition of anelastic performance point (APP).

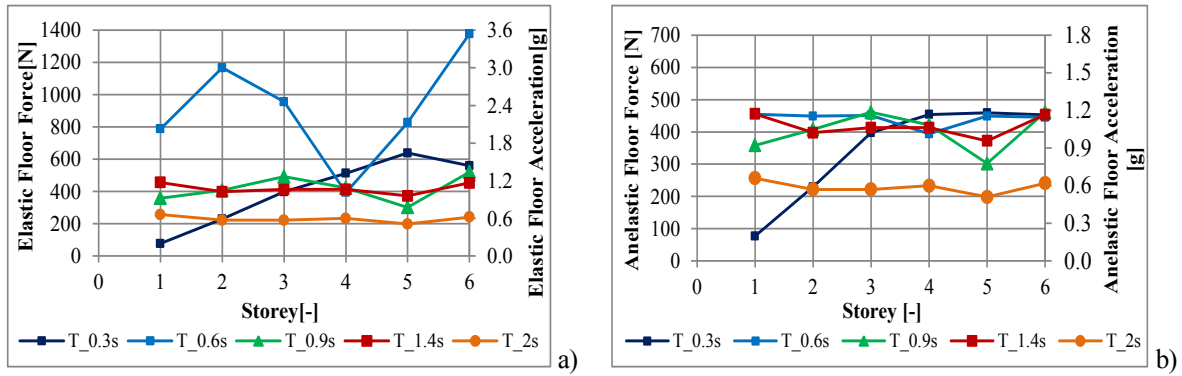


Fig. 83: Force and acceleration acting on suspended ceiling system (hypothesis of equality of displacement):
a) elastic case, b) anelastic case.

The parameters in Table 40 were used to evaluate analytically the seismic force according to the formulation provided by NTC08 (2008) §7.2.3:

$$F_a = \frac{S_a W_a}{q_a} \quad (9)$$

$$S_a = \alpha S \left[\frac{3(1 + Z/H)}{1 + (1 - T_a/T_1)^2} - 0.5 \right] \quad (10)$$

Where:

- W_a is the weight of the non-structural element;
- q_a is the behavior factor (assumed equal to 1, i.e. elastic behavior of non-structural element);
- S_a is the acceleration on the non-structural element defined in (10);
- Z/H is the ratio between the height of the floor at which the element is installed and the height of the building;
- T_a/T_1 is the ratio of the vibration period of the non-structural element and the building;
- S is the coefficient related to the soil category and the topographic coefficient;
- α is the ratio between the maximum acceleration of type A-soil and the acceleration of gravity g ;
- α and S are the same used for the evaluation of the seven spectrum-compatible accelerograms adopted in the numerical analyses.

The period of the suspended ceiling was defined using its initial elastic stiffness evaluated from the bi-linear curve (Fig. 52) and reported in Table 31.

Table 40: Parameters for the definition of floor analytical seismic force acting on each bracing system.

α	S	K_y	T_a	W_a	q_a
[g]	[-]	[N/mm]	[s]	[kg]	[-]
0.103	1.5	35.14	0.209	39	1

The comparison between numerical (N elastic and anelastic) and analytical values with $q_a=1$ (NTC08- 2008) is represented in Fig. 84.

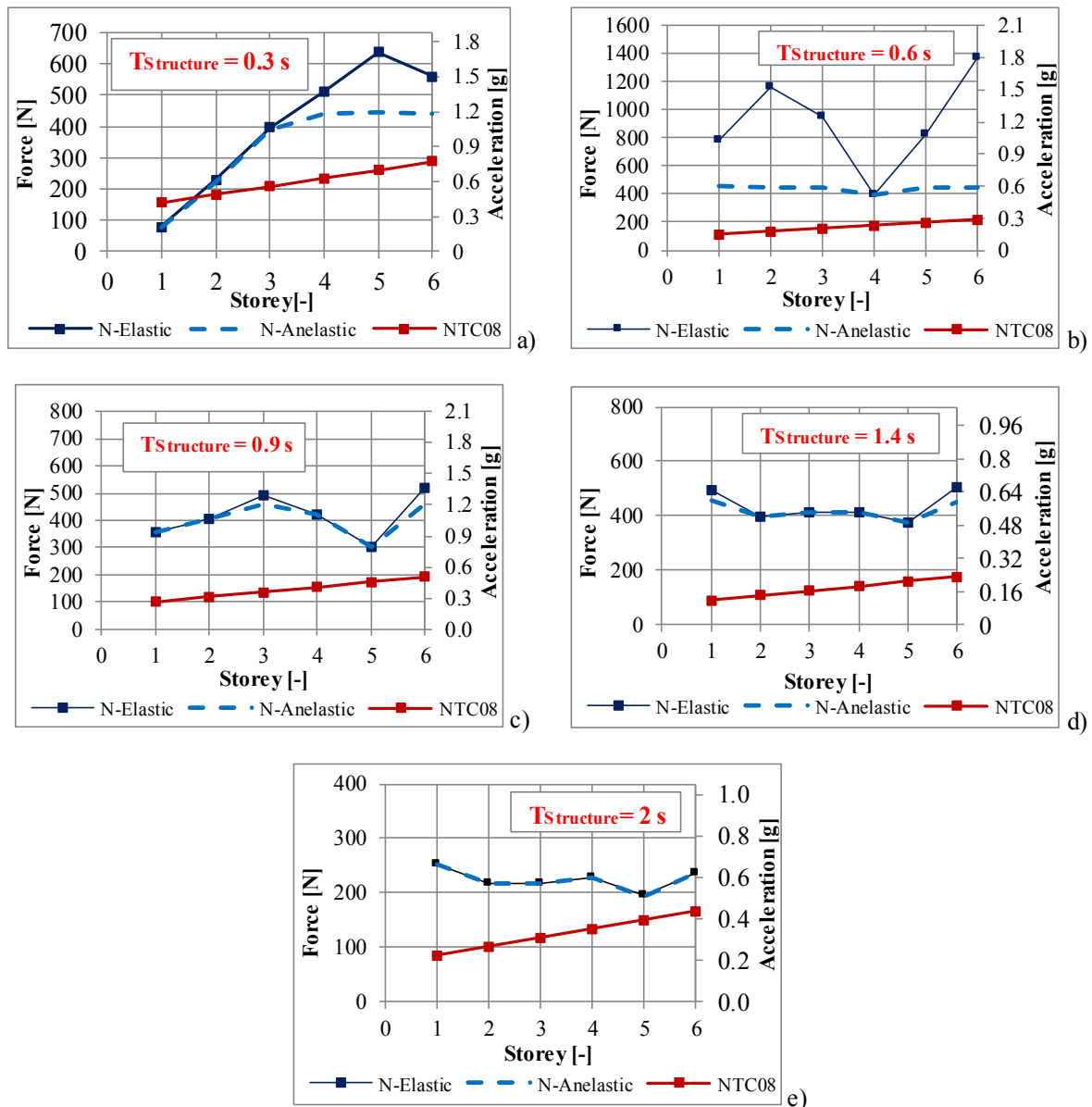


Fig. 84: Numerical-analytical comparison of the floor seismic forces on each seismic bracing system.

The graphs highlight that the analytical formulation underestimates the seismic action and displacement demand on the suspended ceiling except for the first floor of the building with period 0.3 s (Fig. 84 a).

The analytical formulation has a linear growing trend as the floor increases: it reaches a maximum value of 268 N at the last floor of the 0.3 s-period building (Fig. 84 a). As the building period increases, the analytical values of floor force decrease to a value of 167 N for the last floor in Fig. 84 e).

Otherwise, the numerical forces exhibit irregular trends with floor levels. Peaks at certain floors are caused by resonance phenomena with higher order vibration modes that the analytical formulation cannot capture. In analogy with the analytical formulation, numerical method provides lower values of force for higher period-buildings: the minimum value is 237 N at the sixth floor of the building with period of 2 s. But such numerical prevision is 1.42 times higher than that obtained analytically.

4.3.1 Behaviour factor q

The results obtained in the previous section allow to estimate the q -factor for the suspended ceiling as the ratio between the elastic and anelastic force values (Table 41). The hereafter reported values apply specifically for the seismic bracing of the T-Grid system with Metal Tiles, considering a deputy mass of 28.8 kg. The other hypotheses consider the suspended ceiling installed in buildings with different vibration period, located in Amatrice (Rieti – Italy). Obviously such values would change varying strength capacity of bracing system and acting seismic forces.

Table 41: Numerical behaviour factor.

T_{Build} [s]	q Floor 1	q Floor 2	q Floor 3	q Floor 4	q Floor 5	q Floor 6
0.3	1.00	1.00	1.00	1.13	1.39	1.24
0.6	1.73	2.60	2.12	1.00	1.84	3.09
0.9	1.00	1.00	1.07	1.00	1.00	1.13
1.4	1.08	1.00	1.00	1.00	1.00	1.11
2	1.00	1.00	1.00	1.00	1.00	1.00

The results outlined that for this particular case-study the elastic resistance of the suspended ceiling is exceeded only in few floors. A wide plastic excursion is identified only at the Floor 6 in the suspended ceilings placed in the structure with period of 0.6 s (q -factor=3.09).

When the yielding resistance of each bracing system is reached, the seismic forces are cut shifting the ceiling from the resonance condition (Fig. 84 a) and b). The high ductility of the suspended ceilings demonstrated by tests assures the suitability of such q -factor values. The q -factor capacity of the investigated bracing system is higher than the value of $q_a=2.0$ suggested by the NTC08 (2008) (Table 7.2.I) for suspended ceiling systems.

4.4 Conclusions

The results obtained through the numerical analyses using as input the experimental data for the T-Grid system with Metal Tiles, highlighted some shortcomings of the current approach for the calculation of seismic action on suspended ceilings. As already demonstrated by Calvi (2014) and here confirmed, the formulations provided by EC8 (CEN 2013) and NTC08 (2008) do not adequately take into account the resonance phenomena between the main structure and

the secondary element. These formulations also often underestimate the seismic action on secondary elements and consequently their displacement demand.

Further experimental campaigns and numerical simulations will allow to increase the information on the seismic behavior of the suspended ceilings. In this way it will be possible to define some mechanical parameters e.g. the elastic period of vibration, ductility and q-factor for a more accurate evaluation of the seismic action on suspended ceilings.

Furthermore, nonlinear analyses are needed to better understand the interaction between main structure and non-structural elements allowing more realistic evaluation of the displacement demand than the equal displacement hypothesis here assumed. These objective represents the first step toward a displacement-based approach to the seismic safety assessment of suspended ceilings.

Conclusions and future developments

The starting point of this work is a critical study about the State of the art regarding the seismic behaviour of suspended ceilings. The detailed report of the damage observed after past earthquakes allowed to evaluate the weakest details and bad installation techniques of suspended ceilings. The approach used worldwide to enhance the behaviour of suspended ceilings subjected to earthquakes is based on insertion of seismic lateral bracings, whose configuration is prescribed by Standards only in USA.

Shake-table tests are the most common instrument to evaluate experimentally the seismic behaviour of suspended ceilings, often having as only purpose the certification of the product for a certain seismic level. This methodology has many shortcomings, including the high cost and the strong dependence of the results on the chosen seismic input. Regarding the numerical and analytical approach, some numerical models are proposed in literature, but they are meant to characterize only a certain typology of suspended ceilings and can be difficultly used by designers and engineers. Moreover, the formulation proposed by Codes to compute the seismic force on non-structural elements is criticized by some authors as it underestimates the force on them.

Despite of their limitations, experimental campaigns on real-scale suspended ceilings and components are the most effective, and therefore essential procedure to assess the actual seismic performance of suspended ceilings. Consequently, the work in this thesis was addressed toward the experimental seismic assessment of suspended ceilings, through the development and application of an innovative methodology for cyclic quasi-static tests of real-scale suspended ceilings.

The seismic evaluation started with the assessment of the behaviour of inner joints, since they are reported to be one of the weakest components of suspended ceilings. In particular, the behaviour of innovative seismic joints was compared to standard joints and the influence of the shape of the runner to the resistance of the joints was also investigated. The behaviour of the same joints installed in different runners has only a slight difference, that can be neglected. The results showed that neither of the investigated joints has a dissipative behaviour, since the actual seismic approach in the joint design is focused on providing them with strength than ductility.

To perform the tests on real-scale suspended ceiling specimens an innovative test rig was designed and built. Six different typologies of full-scale specimens were tested, varying from T-Grid suspended ceilings to other metal ceilings and finally drywall suspended ceilings. All the specimens have in common the installation of the seismic bracing system formed by four inclined studs connected with bracket to the main runner and the soffit. They represent an alternative bracing system installed in Europe, which differ from that imposed by US standard for the lack of the vertical strut provided with adequate compression strength. In fact, the results and the damage observed are controlled by the buckling mechanism of the studs that resulted quite ductile but unpredictable. Buckling of the inclined studs has caused the lifting and then bending deformation of the main runner with consequent collapse of a tile in the metal ceiling specimens. The results reported in this thesis pointed out the limitations of the bracing system here studied. On the other side, the lateral bracing system prescribed by the US standards seems to assure higher performances since, if adequately designed, it should avoid any vertical dislocation of the suspended ceiling, therefore preventing the inflection of the ceiling and the

fall of panels. However, as the two bracing systems haven't been both tested through cyclic quasi-static tests, it's not possible to make a direct comparison of their effect to the global seismic behaviour.

In order to evaluate the relation of the void height on the resistance of the seismic bracing and the observed damage, it was varied during the campaign. Analytical evaluations on the buckling critical load were performed and indications for the design of the seismic bracing systems are presented.

Other details affecting the final resistance were reported to be the inclination of the studs of the bracing system and the connection of the bracing to the main runner. But the reduced number of specimens tested for each combination of void height and suspended ceiling typology wasn't enough to clearly define the influence of each factor.

Also it wasn't possible to assess the influence of the joints tested on the overall behaviour of suspended ceilings. In fact, the particular typology of joints evaluated experimentally was installed only in the T-Grid ceilings and during the tests no damage on the joints was reported. This result can be explained with the over resistance of joints than the buckling of the bracing studs.

With the aim of generalizing the experimental results obtained, the Floor Response Spectrum method was applied to six-floor buildings located in the high seismic zone of Amatrice (Rieti – Italy). The case-study buildings have five different periods of vibration and include the installation of T-Grid suspended ceilings with Metal Tiles at each floor. The force and displacement demand on the suspended ceilings were compared to their capacities. The elastic floor spectra and the capacity curve were plotted in the Acceleration Displacement Response Spectrum Domain and the results were then compared to the procedure proposed by the European/Italian Code. The results outlined that the analytical formulation proposed by Codes underestimates the seismic force and displacement demand on suspended ceilings. Moreover, the analytical formulation has a linear growing trend proportional to the height of the building that cannot describe the resonance phenomena outlined by the numerical results.

With the assumption of equal displacements in case of elastic and anelastic behaviour of suspended ceilings, the q-factor value has also been estimated for each floor in each case-study. The suspended ceilings remain mainly on their elastic behaviour except for a wide plastic excursion at the last floor for the building with period of vibration equal to 0.6 s, resulting in a value of q-factor equal to 3.09. However, the experimental results outlined the high ductility of the ceilings considered and thus the suitability of the q-factor values reported.

The approach adopted in this work overcomes the limitations of the Code formulation, moreover it is sufficiently simplified but accurate to be adopted by engineers and suspended ceilings manufacturers in a seismic design context.

Further future improvements to the work presented in this thesis could regard both the experimental and the numerical aspects.

The experimental setup can be modified with the installation of a vibrodyne in the CLT ceiling in order to apply a dynamic vertical input to the suspended ceiling sample. This modification will allow to investigate the behaviour of suspended ceilings subjected to dynamic vertical loading, that is one of the main cause of collapse according to the experimental results presented in Literature. Moreover, other types of suspended ceilings and seismic bracings available in the market can be tested with cyclic quasi-static tests, including the technology prescribed by the American standards. These tests can allow accurate comparisons regarding

their seismic behaviour and will contribute to create a database with the capacity curves of suspended ceilings used worldwide.

Finally, the factors affecting the seismic behaviour of suspended ceilings can be investigated in an extensive experimental campaign. Characteristics to be considered are the void height, characteristics of the seismic bracing, the dimension of tiles and the loading direction in relation to the main runner direction.

From a numerical perspective, a displacement-based approach will be the further step for the seismic evaluation of suspended ceilings and will require nonlinear analyses to be performed in order to identify the displacement demand of suspended ceilings.

A final consideration regarding the lateral bracing elements that are commonly chosen as seismic solution for the suspended ceilings is presented. These systems have proved to be difficult installed, due to the presence of utility systems attached to the soffit and limited in providing seismic resistance. Future research should focus on the design of a new technology that can allow the suspended ceiling to move freely but be able to limit lateral displacement and pounding effects.

Acknowledgement

"If I have seen further, it is by standing upon the shoulders of giants" (Isaac Newton). This thesis has been developed with the guide of Roberto Scotta, whose broad shoulders and brilliant brain are guiding a lot of students every year to become better engineers and researchers. I can proudly say that I am one of them.

I wish to thank Sara, my right hand, to whom this thesis is dedicated. Her intelligence, strength and patience have been essential for the work developed in this thesis.

I would like to thank Davide, the person who patiently taught me all I know about experimental and numerical analysis. I'd like to thank also "buon" Luca and Lorenzo for their punctual explanations, brilliant ideas and friendly support.

I wish to thank all the research group, starting from my first office colleagues Davide Gira and Michela to the newest acquisitions Valentina and Giovanni. I want to thank also Enrico, Massimiliano, Luca, Anna, Luisa, Diego, Vit. Finally I would like to thank my friends from Lunetta office: Luca, Sebastiano and particularly Irene.

I would like to thank Nicolò and Enrico for the suggestions and the help with the choice of the experimental instrumentation.

I also want to thank the staff of the Construction materials testing laboratory for the support with the tests on suspended ceilings joints.

I'm grateful to Fassa Bortolo srl and Atena spa. A special thank you goes to Monica, who coordinated the efforts during the experimental campaign. I also want to thank Martino, Alberto and Piero for the help with the design of the experimental setup. Finally I would like to thank Simone, Alvis, Rommi and Alessandro for the contribution in the realization of the experimental setup and the optimization of the technical details.

I'm grateful to my beloved parents, Stefania and Silvano, my first supporters and models. I would like to thank Lucio, the person I am most proud of. I wish to thank all my big family, for the suggestions, the encouragement and the love.

I want to thank all my friends and in particular my milestones: Anna, Chiara, Cristina, Emma, Giovanna.

Finally, the suspended ceilings has brought me into a new adventure. I'm grateful to all the people I've met until now, who tried their best to make me feel welcome.

I would like to thank Matt for reminding me that I would have surprised myself.

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