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Federico M. Mazzolani
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Proceedings of the 11th International Conference on Behaviour of Steel Structures in Seismic Areas

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Foreword

The international series of specialty conferences on Behaviour of Steel Structures in Seismic Areas (STESSA) continues to bridge the gap between scientific research, design codification and practical applications in the field of design and assessment of seismic-resistant steel structures. Since its beginning in Timisoara, Romania, in July 1994, STESSA has served as a vital international forum, enabling researchers and engineers to share and discuss the latest developments in this specialized area.

Following its successful launch, STESSA conferences have been hosted every three years across the globe: Kyoto, Japan (1997); Montreal, Canada (2000); Naples, Italy (2003); Yokohama, Japan (2006); Philadelphia, USA (2009); Santiago, Chile (2012); Shanghai, China (2015); Christchurch, New Zealand (2018); and returning to Timisoara, Romania, for its tenth anniversary in 2022. Each edition has drawn participants worldwide to explore advances in design, assessment and testing of steel structures, overcoming challenges and learning from each seismic experience, which is unique in each location.

The eleventh edition of the STESSA conference is continuing this important tradition. Scheduled for 2024 in Salerno, Italy, this venue choice reflects STESSA's ongoing commitment to covering major earthquake-prone regions of the world, from Southern Europe to Pacific Asia, the American continent and Oceania. The selection of Salerno also marks a recovery and a symbolic return to the regular triennial scheduling after the year lost due to the pandemic.

As in previous years, the STESSA conference in 2024 has gathered leading academics, researchers and engineers to discuss the myriad of facets in the field of steel structures in seismic areas. The conference addresses both traditional and emerging topics, such as the behaviour of structural members, connections and systems; mixed and composite structures; energy dissipation, self-centring and low-damage systems; as well as assessment, retrofitting, codes and standards. During the conference, 195 scientific contributions and five keynote lectures are planned to be presented.

The eleventh edition of the STESSA conference is jointly organized by the Department of Civil Engineering of the University of Salerno and the Department of Structures for Engineering and Architecture of the University of Naples "Federico II", in cooperation with the CTA *"Collegio dei Tecnici dell'Acciaio"*.

We greatly appreciate the continuous support of the Technical Committee 13 "*Seismic Design*" of the European Convention for Constructional Steelwork (ECCS). Our sincere thanks also goes to the University of Campania "Luigi Vanvitelli," the "Fondazione Promozione Acciaio", the "Istituto Italiano della Saldatura", the "Associazione Italiana di Metallurgia" and all the sponsors for their invaluable support.

On behalf of the organizing committee of STESSA 2024, we express our profound gratitude to all keynote speakers, authors, as well as to the members of the international scientific and advisory committees. We also extend our thanks to all the participants in this important international scientific event.

Special recognition is due to the publisher, Springer Nature Switzerland AG, particularly to Pierpaolo Riva, for his unwavering support in publishing the STESSA 2024 proceedings. This contribution significantly enhances both dissemination and impact of our collective work on seismic-resistant steel structures. We are committed to continuing this tradition of excellence and innovation in the study and application of steel structures in seismic areas.

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Contents

The Resiliency of Steel Moment-Resisting Frame Structures Against Earthquake: The FUTURE Project

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Abstract. The article presents the main experimental findings of the FUTURE project that was funded in the framework of the H2020-INFRAIA SERA program. Shake table tests of four steel mockups were carried out. The tested mockups were two-story, single-bay moment frames equipped with replaceable dissipative elements at the beam-to-column joints and column bases. Three mockups were tested varying the type of prequalified beam-to-column joints (i.e., Reduced Beam Section, Extended Stiffened End-Plate, and Haunched end-plate). All specimens were equipped with Reduced Column Sections (RCS) at the column bases. One mockup integrated ductile infill wall and suspended ceiling to explore their effects on the structural performance. Each mockup underwent more than thirty incremental near-field (NF) ground motions arranged also to simulate foreshock-mainshock-aftershock scenarios. The study reveals that despite the severity of the excitation regimes, all mockups exhibited satisfactory performance, confirming the effectiveness of the design criteria. Besides, the type of beam-to-column joints significantly affects the behavior of SMRFs in terms of the maximum and residual inter-story drift ratios. It is also shown that while the ceiling system remained almost intact during the excitations, the infill walls experienced higher accelerations than the expected values and damage due to Wall-Frame Interaction.

Keywords: Steel Moment-Resisting Frame · Beam-to-Column Joint · Column Base · Dissipative Elements · Nonstructural Components

1 Introduction

Steel Moment-Resisting Frames (SMRFs) are versatile lateral force-resisting systems thanks to their architectural flexibility and high seismic ductility. Previous severe earthquakes revealed the high vulnerability (e.g., considerable joint damage) and low resiliency (e.g., extended repair time) of SMRFs in the case of their poor design [1–3]. Besides, many previous investigations outlined the significance of the beam-to-column joint behavior, the type of ground motion, and the presence of nonstructural components on the seismic performance of SMRFs [1–5]. Most of the existing experimental studies on SMRFs typically involved quasi-static tests on single components (beams, columns, or their assemblies) or dynamic shake-table tests on either the whole or portions of a

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frame structure. While the latter offers valuable insights into the actual behavior due to replicating a real seismic event, their considerable expense and effort have limited the number of such tests on SMRFs. Moreover, as detailed in the following section, many shake table tests were conducted on structures designed according to a specific regional code and technological practice (e.g., US, Japan, and China) that largely differ from those applied in other regions (e.g., Europe).

In addition to the progress of scientific research, design codes are also evolving, often in parallel to many studies contemporary running, which often makes unfeasible the implementation of the most updated knowledge. In Europe, many efforts have been made to amend the structural Eurocodes since 2015. In this regard, the second generation of Eurocode 8 (which governs the seismic design of structures) widely differs from its predecessor [6]. Regarding SMRFs, a set of prequalified beam-to-column joints has been introduced on the basis of [7, 8]. All joints uniformly employ bolted end plates, eliminating the need for field welding. These joints can be alternatively designed to restrict the plasticity into either beams or the end plates without yielding in the panel zones. This also increases the structure's resiliency by facilitating the replacement of the damaged elements following a severe earthquake. Nevertheless, despite their resemblance to their North American counterparts, recent studies revealed their different behavior concerning strength and ductility [9, 10]. Such observations stem from quasi-static experiments on isolated beam-to-column assemblies, necessitating further evaluation within whole 3D structures and under realistic seismic scenarios.

The design of SMRFs is usually governed by stiffness rather than strength requirements. The building codes' restrictive drift limits often enforce greater beam sizes than those specified by the strength design, leading to larger sections for other members in line with the capacity design requirements (e.g., weak-beam-strong-column). In the 2^{nd} generation of EC8, the interstorey drift limits and the control of P-Delta effects have been substantially revised, thus permitting more flexible structures than those compliant with the 1^{st} generation of EC8 [4, 11]. Therefore, in order to minimize the detrimental consequence of the structural interaction with ancillary elements (e.g., facades, partition walls, etc.), the ductility and deformation capacity of these system should be qualified to accommodate larger interstorey drift ratios. In this regard, a series of tests have been conducted on a set of ductile ceiling and drywall infill solutions patented by the Knauf company [12–14]. The results showed promising outcomes by delaying the incipient of crack initiation and minimizing damage from low-intensity and moderate excitations. Nonetheless, these results were solely obtained using quasi-static or dynamic tests of isolated ceiling and wall solutions without considering their interaction with the structural members.

All the above-discussed considerations motivated an experimental program in which shake table tests were conducted at the CEA (Saclay, France) on four steel moment-resisting frame structures having different types of beam-to-column joints, with one mockup also incorporating nonstructural components. This experimental campaign was framed within the European research project "Full-scale experimental validation of steel moment frame with EU qualified joints and energy-efficient clad-dings under Near fault seismic scenarios" (FUTURE), which was funded within H2020-INFRAIA SERA framework [15].

In the following sections, first, the details on the geometry of the mockups, sensor distribution, lab facility equipment, and ground motion records are presented. Later, after an overview of the observed structural and nonstructural damages, the responses of each mockup are explained in terms of maximum and residual inter-story drift ratio, floor acceleration, and maximum acceleration ratio in nonstructural components.

2 Experimental Mockup

The mockup was two-thirds scaled from its initial size to comply with the shake-table limitations. It is a two-story, one-bay SMRF that was basically design according to the next generation of the Eurocodes. It consisted of two perimeter SMRFs in the loading direction and two Concentric Brace Frames (SCBFs) in the orthogonal one that were solely applied to prevent any potential torsional rotations of the floors. All dissipative elements (i.e., the beam ends and its connections, and the lower segment of the first storey columns) were made of S235 JR steel grade, while the remaining components were of S355 JR steel to confine yielding solely into the dissipative elements. Bolted flange and web plate splices were incorporated to facilitate the replacement of the damaged parts after each round of excitations. The structure supports two-segment 150mm thick slabs at each story level with area reductions in each corner to facilitate access and inspection of the dissipative beam parts. Designing the slab in two parts was necessary to ease lifting and assembling the slab in the CEA laboratory. Besides, two additional 65 mm thick steel plates were placed on each slab to increase the seismic masses further. Figure 1 illustrates the details of the mockup components.

As depicted in Fig. 2, three different beam-to-column joints and a unique column base detail were tested. The beam-to-column joints were of Reduced Beam Section (RBS), Haunched, and Extended Stiffened End-Plate (ESEP) types recently qualified in Europe. The first two types were designed to shift the plastic hinge away from the column face, while the third one was designed to allows for the simultaneous yielding of the unstiffened portion of the bean and the end-plate connection, thus reducing the plastic rotation demand in the beam. All connections were replaced after each round of excitations. The panel zones were designed to behave elastically to further increase the repairability (i.e., the resiliency).

Regarding the lower segments of the columns at the base level, a Reduced Column Section (RCS) geometry was implemented in order to (i) reduce the demand on the base plate connection and (ii) facilitate the formation of the frame sway mechanism.

The first-story ceiling comprised hangers attached from the slab underside, linking to a grid of main and cross runners. Gypsum panel boards were attached to the grid by self-screw bolts. Braces were not included, and the hangers were designed to resist both vertical and horizontal seismic forces. Regarding the infill walls, a supplementary framing system of Vertical and Horizontal Boundary Elements (HBE and VBE) has been considered to reduce the Wall-Frame Interaction (WFI). The frontal facades (i.e., those orthogonal to the loading direction) were connected to struts at midpoints and VBE at the extremities. On the other hand, the side walls (i.e., those parallel to the loading direction) were connected to struts in the middle and directly to the columns at the ends. Figure 3 depicts the details of the ceiling and the infill walls.

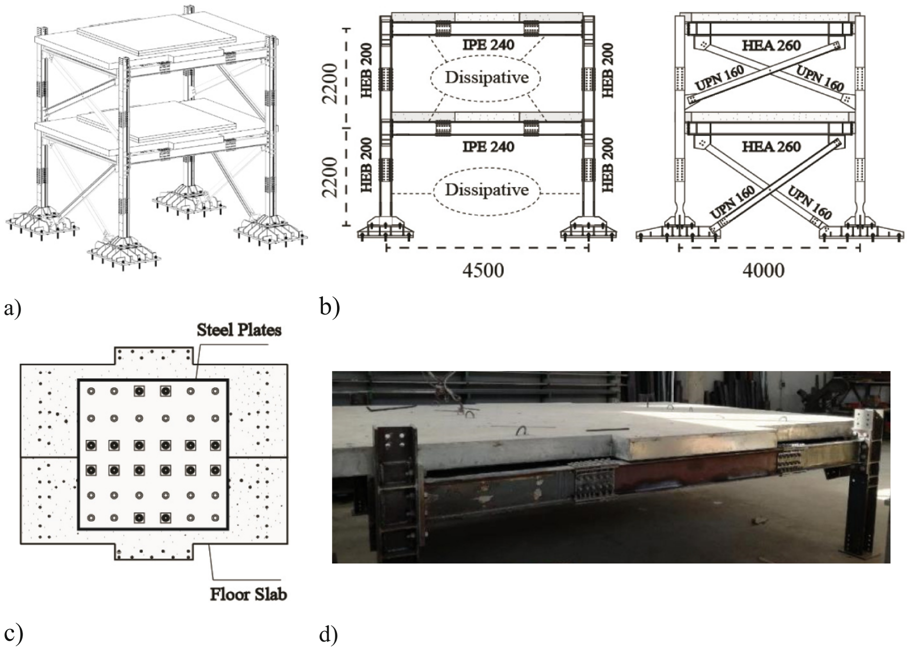


Fig. 1. Mockup geometry: (a) 3D view; (b) side views; (c) plan view of the slab; (d) details of the mounted slab.

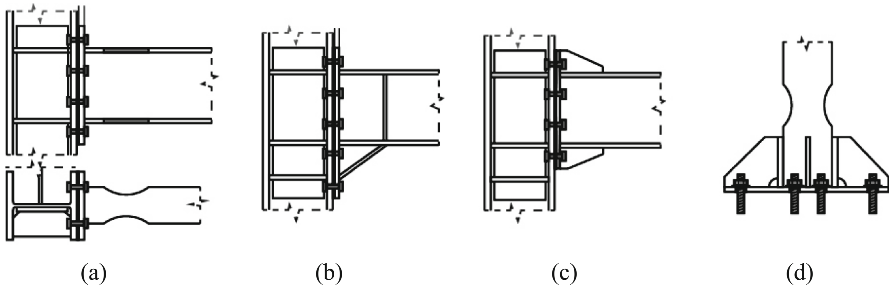


Fig. 2. Mockups' Beam-to-column connections and column bases: (a) RBS, (b) Haunched, (c) ESEP, and (d) RCS.

It is also important to outline that the mockup equipped with non-structural elements (here and after referred to as infilled) also employed the RBS beam-to-column joint type and RCS at column bases.

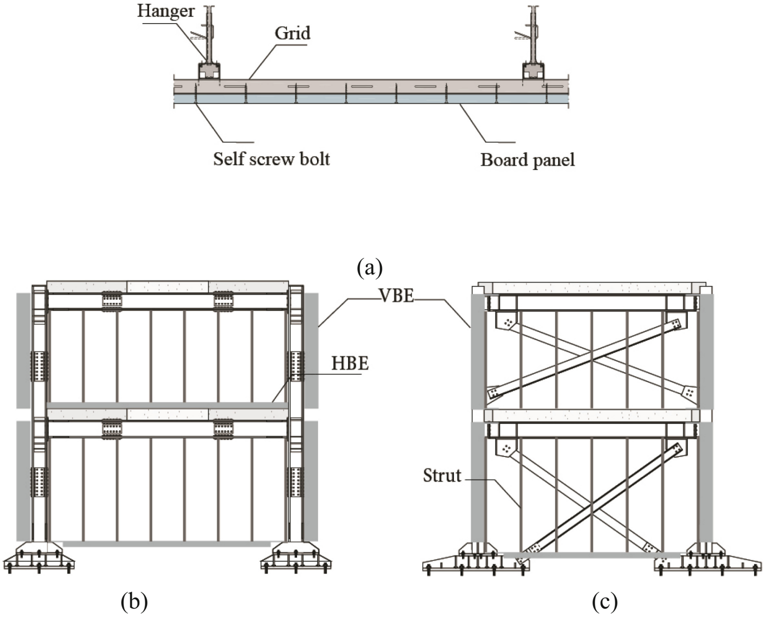
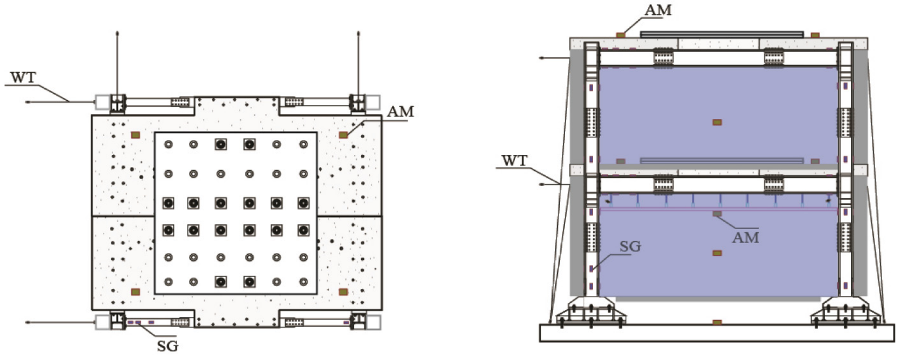


Fig. 3. Details of nonstructural elements**:** (a) ceiling, (b) side walls, and (c) frontal walls.

3 Distribution of the Sensors and Measuring Devices

Figure 4 describes the layout of the sensors and measuring devices that were installed on the mock-up to measure its experimental performance. The layout of the instrumentation is composed of about 120 measurement channels from displacement, velocity, acceleration, and deformation sensors. As shown in Fig. 4, Wire Transducers (WTs) were utilized to record the story displacements. Multiple Strain Gages (SGs) were attached to the joints and along the members to estimate the internal forces. Linear Variable Displacement Transducers (LVDTs) recorded the extent of joint end-plate and column base-plate rotation. On each floor, AcceloMeters (AM) were placed at each corner near the columns to record the floor acceleration and check for the integrity of the slabs during excitations. Multiple AMs were also attached to the ceiling surface and infill walls to capture their in-plain and out-of-plain accelerations. Figure 5 clarifies how the measuring instruments were applied to the mockup.



(a) (b)

Fig. 4. Mockups' Sensor Distribution: (a) plan view, (b) elevation view.

Fig. 5. The instrumentation and measurement devices installed on the mock-up.

4 Shake Table, Ground Motions, and Testing Procedure

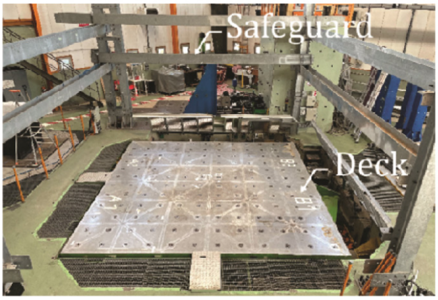
The tests were conducted on AZALEE shake table at the CEA laboratory in France. This table consists of a 6 m × 6 m aluminum deck connected to a set of actuators, exciting the table in horizontal and vertical directions. A Safeguard framing system surrounds the table to eliminate damage to laboratory equipment.

Two different natural records were selected, namely: (i) the Northridge earthquake (January 17, 1994) recorded at the Newhall Fire station (referred to as NO2–70); (ii) the Kocaeli earthquake (August 17, 1999), recorded at Izmit (referred to as ST553_Izmit). These input ground motions have been selected for the following reasons:

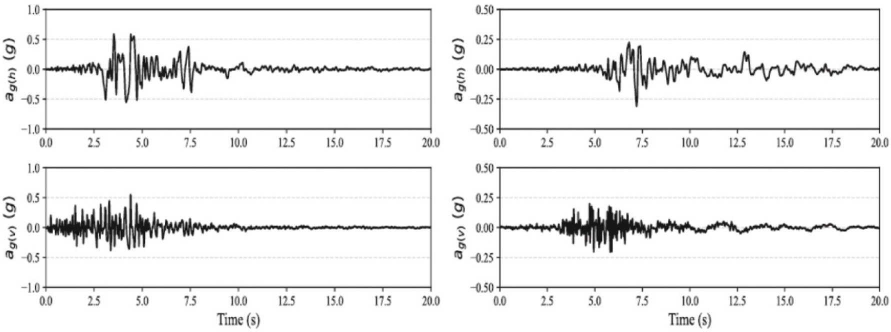
1) They are representative of Near-field ground motions;
2) Both longitudinal and vertical accelerations have similar magnitude, thus facilitating the scaling of both components with the same factors;
3) On the basis of the results of predictive time history analyses [16], the overall performance of the mock-up subjected to these strong motions at near col-lapse limit state is compatible with the capacity of the shake table in terms of overturning moment and base displacements.

The selected earthquake records were scaled in the time domain by a factor equal to 0.816, since the mock-up is 2/3 scaled from the reference building, in accordance with the criterion given in [17]. In addition, the tests were carried out scaling the acceleration from 0.1g to 1.7g.

Figure 6 (a), (b), and (c) illustrate the shake table as well as the vertical and horizontal components of the Northridge and Kocaeli ground motions.



(a)



(b) ©

Fig. 6. The shake table in the lab (a), (b) Northridge, and (c) Kocaeli records (horizontal "h" and vertical "v" components).

Two different natural records were adopted for the shake table tests, namely: (i) Northridge earthquake (January 17, 1994) recorded at the Newhall Fire station (referred as NO2–70); (ii) Kocaeli earthquake (August 17, 1999), recorded at Izmit (referred as ST553_Izmit). These input ground motions have been selected because of the following reasons:

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The selected earthquake records were scaled in the time domain by a factor equal to 0.816, since the mock-up is 2/3 scaled from the reference building. In addition, the tests were carried out scaling the acceleration from 0.1g to 1.7g.

Each mockup was subjected to more than 30 excitations. While having a generally increasing trend, the excitations were also scaled to represent foreshock-mainshock-aftershock sequences. Before and after each sequence, white noises were imposed on the mockup to measure their damping ratio, vibration periods, and associated modal shapes. After each round of excitations, the dissipative elements (beam ends and column bases) were replaced. Figure 7 shows the assembled bare mockup with RBS beam-to-column joints and the mockups with non-structural elements.

(a) (b)

Fig. 7. Examples of the assembled mockups: (a) bare frame with RBS joints, (b) infilled.

5 The Damage Pattern in the Primary Elements

Figure 8 summarizes the damages experienced by the beam-to-column joints and column bases after the end of the imposed excitations.

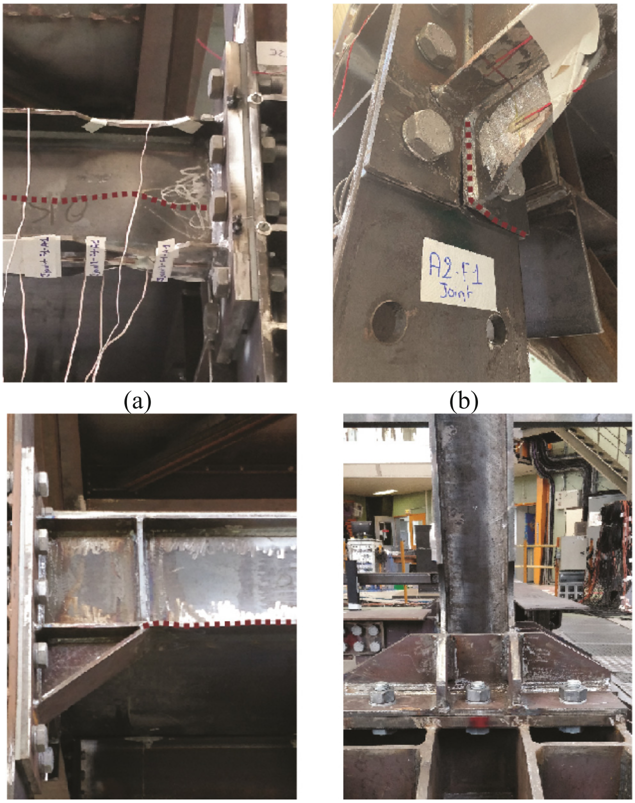
As a general comment, it is observed that the presence of the transverse beams did not appreciably affect the response of the primary moment resisting joints.

The damage of RBS joints is characterized by severe Flange Local Buckling (FLB) and Web Local Buckling (WLB), resulting in out-of-plane displacement at the second-story beams within the plastic hinge region. However, the RBS joints in the first story showed moderate WLB.

In the case of the ESEP connections, post-test observations revealed the end-plate yielding and base-metal fracture around the stiffener-to-endplate welds. Although yielding occurred, no evidence of FLB or WLB were noticed in the unstiffened segment of the beam.

Regarding the Haunched joints, only moderate FLB was observed, while residual plastic rotation was evident into the plastic hinges.

Meanwhile, the RCS column bases showed favorable responses in all tested mock-ups. In fact, apart from residual plastic rotation, no flexural buckling, lateral-torsional buckling, FLB, or WLB was observed in the yielded lower segments of the column at the base level. This aspect is interesting because it can open the way to discuss about the possibility to use RCS details in real steel MRFs.



(c) (d)

Fig. 8. Damage pattern of the dissipative zones after the imposed excitations: (a) RBS, (b) ESEP, (c) Haunched, and (d) RCS.

Figure 9 summarizes the damage pattern of the non-structural elements that was observed after the all applied excitations. As it can be observed, the ceiling remained almost intact during all imposed excitations, as shown in Fig. 9(a) and (b). In fact, except for the loosening of some panel boards at the center of the ceiling, which is ascribable to the interaction with the attached accelerometers, no significant damage in terms of panel crack, yielding of the grid, and separation of the hangers for the slab was observed.

In the case of facades, the response of the panels perpendicular to the horizontal excitations differed from those exhibited by the other panels. The frontal facades (i.e., perpendicularly placed to the direction of loading) did not exhibited any significant damage. In fact, no loosening, separation, or hanging of the panels were observed, solely some minor vertical cracks close to the VBEs were evident at the end of all imposed signals. On the contrary, the infill walls in the loading direction showed vertical cracks, fractures, and separation of the panel board at their ends.

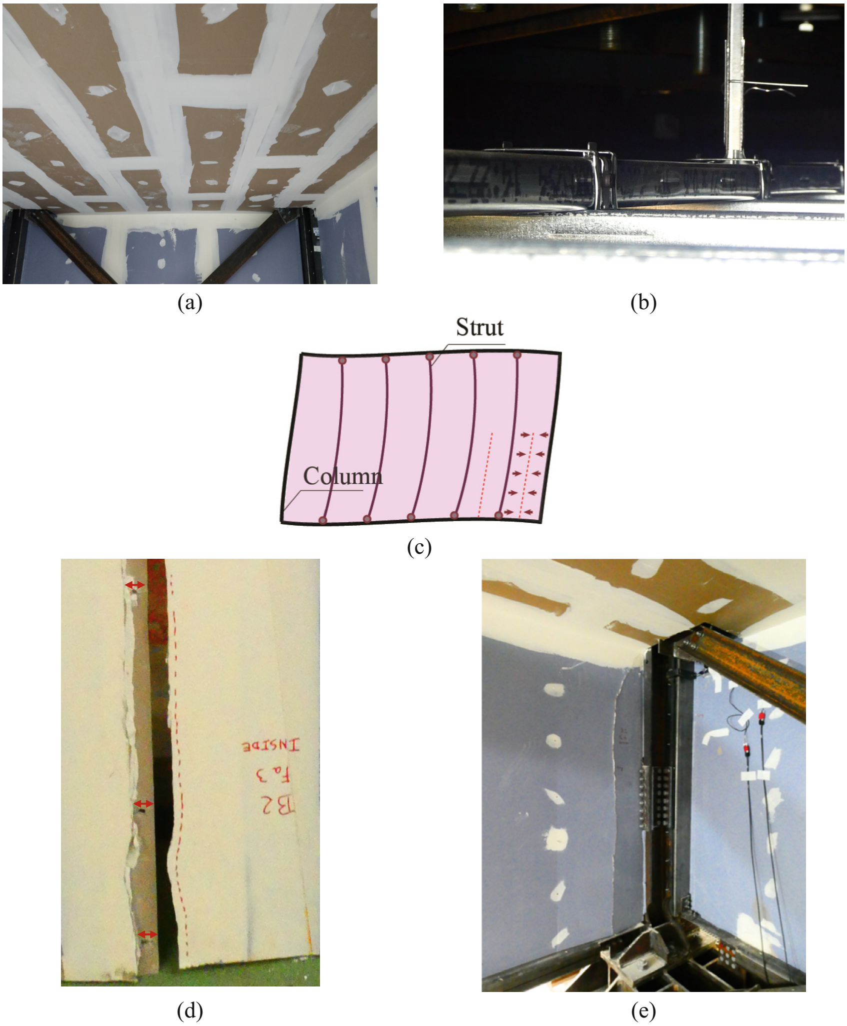


Fig. 9. Damage of nonstructural elements: (a, b) ceiling, (c) mechanism of the infill wall, (d) (d) horizontal cracks at the bolt holes, and (e) vertical cracks close to the panel and column.

These phenomena mainly occurred due to the difference in the deformed shapes of the wall struts and the beam-column assemblies, leading to the WFI at the end panel boards. The mechanism driving this behavior is illustrated in Fig. 9(c), which showcases the interaction between the beams and columns of the SMRF and the panel boards of the non-structural infill wall. As shown, the columns deform in double curvature (fixed-fixed) due to the seismic forces, while the struts undergo single curvature (pin-pin) deformation resulting from the inertial forces of the panel boards. This distinction