



Curriculum 3. Modelling and Simulation

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**Progressive Collapse Assessment
of Steel and Concrete Composite
Structures Subjected to Extreme
Loading Conditions**



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and Concrete Composite Structures
Subjected to Extreme Loading Conditions

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Abstract

Accidental events, such as impact loading and explosions, are rare events with a very low probability of occurrence, but their effects often lead to very high human losses and economical consequences. Vulnerability of structures to the effects of local damages and its mitigation are issues widely discussed inside the scientific community. The structural property associated with such a vulnerability is named robustness. Depending on the type of the structural system and on the importance of consequences, specific design strategies can be adopted in order to ensure a robust structural response. Among them, the system redundancy, the joints and members ductility and the alternate load paths are the ones commonly adopted in case of multi-storey framed buildings. The present work focuses on the study of the behaviour of steel-concrete composite structures subjected to a column loss, and proposes a global overview to quantify the robustness of such systems subjected to this hazard scenario. The description of validated finite element models and of a new analytical tool to predict the response of flat concrete slabs subjected to large displacement are reported in this dissertation. Furthermore, important design hints for composite buildings are proposed. The starting point of the research is an experimental campaign conducted at the University of Trento. Two tests on 3D full-scale one storey composite steel-concrete frames, extracted from five storeys frames designed in accordance to the Eurocodes, were performed simulating the central column removal. The role of the beam-to-column connections and of the concrete slab for the force redistribution was investigated. The experimental data have been then taken as reference for the calibration of finite element models that allowed to conduct further numerical analyses on different structural configurations and design scenarios. In particular, it was studied the influence of the location of the removed column on the structural behaviour. The collapse of central, lateral and corner columns were investigated in order to understand the load transfer mechanism, the requirement of joint ductility and the influence of the concrete slab on the development of alternate load paths. Both experimental and numerical results showed that the concrete slab plays a key role on the load transfer mechanism within the structure: it can hence contribute significantly to the robustness of the system preventing progressive collapse. The knowledge of the response of reinforced

concrete slabs subjected to large displacements, as in the case of a column loss, allows quantifying the contribution to the resistance of the building to collapse associated with activation of membrane forces. Regarding this aspect, a new analytical simplified method, based on the principle of virtual works, was developed to predict the load-deflection response of simply supported reinforced concrete slabs with planar edge restraints subjected to large displacement. In conclusion, the present work provides a significant contribution to the knowledge of composite steel-concrete structures subjected to extreme loading conditions and open the way to extend results to different structural configurations and loading scenarios.

Chapter 1

Introduction

1.1 Overview

During their life-span, multi-storey buildings are subjected to different types of loading, ranging from live loads, seismic loads, and accidental loads. For each kind of action, the structure should be able to ensure a sufficient level of reliability in terms of strength, serviceability and durability. Due to the unforeseen nature of the accidental actions, e.g. fire, explosions, or impacts, the structure could be subjected in its life to a load not explicitly considered in the design, and the risk of local damage is certainly probable. Similarly, local structural damage can lead to a spread of the failure to nearby parts, causing a progressive collapse of a great portion of the structure, and even to the entire structure, as confirmed by several catastrophic events occurred in the recent years (e.g. Ronan Point, World Trade Center). From the Ronan Point event (London, 1968), the research community started studying the progressive collapse phenomenon, and the design requirement of robustness have been included in several design codes, including the Italian NTC [2008]. To date, the robustness of structures is a topic widely studied by the research community, and important results have been obtained. However, there are various issues yet to be solved.

As stated in the above, accidental actions may cause important damages to a structure, leading to a loss of the structural integrity and of the stability in part or the entirety of the structure. The EN1991-1-7 [2002] defines a robust structure as a structure able to withstand accidental events without being damaged disproportionately with respect to the original cause. Design strategies such as local strengthening of members and developing of alternate load paths can enhance the global response and reduce the vulnerability to progressive collapse. In this context, redundancy and ductility, in particular of the beam-to-column connections, ensure a sufficient level of robustness.

The column removal case is one of the most accepted hazard scenarios to

consider studying the effects that accidental actions cause to a structure. This hazard scenario is also widely used and recommended in design standards and guidelines. The collapse of a column causes to a structure dynamic effects, large deformations of the flooring system and high rotations in the beam-to-column connections. The extension of the associated damage strongly depends on the capability to redistribute loads from the damaged to the undamaged parts and to reach a new stable equilibrium configuration. In this context, the alternate load paths, the ductility offered by the joints and the three-dimensional capabilities of the flooring system constitute essential factors and design strategies for a robust structural response.

1.2 Motivation and objectives

Structural robustness is currently a research topic largely 'popular' among the scientific community. Since 1968, important studies have been conducted and significant results have been obtained. Despite this, the research is still limited and many questions have to be answered.

Designing and analysing framed structures subjected to large displacement as a consequence of a column loss is a challenging task. In recent years, several experimental and numerical studies have been conducted on entire steel, concrete or composite steel-concrete framed structures, sub-structures or single structural members. Even if some experimental campaign were carried out, the current literature focuses mainly on the two-dimensional behaviour of frames, or on the isolated behaviour of the slab by itself. In order to improve the knowledge of the actual frame response, experimental tests on 3-dimensional frames are the most complete and recommended approach. These experiments are expensive and complex, and, to date, few experimental data are available. On the other hand, the experimental data are a vital background to develop and validate numerical models. Finite element analysis and analytical developments allow simplifying complex problems without losing the necessary accuracy of the results.

The importance of improving knowledge of progressive collapse resistance of structures subjected to accidental actions has been previously justified. One area where the knowledge is still inadequate is that of steel-concrete composite frames. Hence, the main aim of this research is to investigate the behaviour of multi-storey composite steel-concrete frame buildings subjected to a column loss. The starting point of the research is an experimental campaign conducted at the University of Trento among the activities of a RFCS European research project named as 'RobustImpact' (European Commission [2012]).

To achieve the aforementioned goal, the following objectives are fulfilled:

- literature review on the main experimental, numerical and analytical studies on structural robustness;

- investigation on the frame response subjected to a column removal by means of experimental and numerical analysis. Definition of a calibrated finite element model to conduct parametric analyses;
- study of the different impact on the structural response of location of the removed column. Finite element models on full-frame structures and extracted sub-frames has been developed and the results compared between them;
- quantification of the slab contribution on the structural behaviour;
- assessment of the principal analytical methods to evaluate the response of slabs subjected to large displacements;
- development of a new analytical method, based on previous studies of Omer [2006] and Herraiz Gómez [2016], in order to investigate static load-deflection response of concrete slabs in the membrane stage.

1.3 Outline of the dissertation

This dissertation focuses on the effects of extreme loading condition in structures and comprises seven chapters to study the column removal effects in composite steel and concrete frame buildings. Moreover, the important topic of the behaviour of a concrete slab subjected to large displacements is discussed and a new approach presented in Chapter 7. After a short overview in Chapter 1 to illustrate the motivation and objectives of the thesis, Chapter 2 "Literature review" provides a description of the principal studies on structural robustness. Both experimental and numerical studies are reported, and current research needs are clearly pointed out. Furthermore, a short description of the main analytical developments for progressive collapse assessment is reported and commented.

Chapter 3 "Robust design of building structures" provides an overview of the principal design Codes for accidental actions, and highlights the necessity to improve the Standards with specific recommendations.

In Chapter 4 "Building structures subjected to column loss" the experimental tests on full-scale sub-frame structures conducted at the University of Trento are presented and the main results reported. The finite element models of the tested structures are described and the comparison of the results with the experimental ones is presented and commented. Furthermore, the contribution of the concrete slab to the structural response is highlighted by comparison of results between finite element analysis conducted with and without the slab.

In Chapter 5 "Effect of location of removed column on structural behaviour", the response of composite steel-concrete frame buildings subjected to different columns removal is studied by means of the previously validated nu-

merical model. In particular, central, lateral and corner columns loss have been simulated. The results allow understanding the force and load redistribution and the associated requirement of joint ductility.

In Chapter 6 "Development of membrane action in concrete slabs" the principal analytical methods carried out in the recent years to evaluate the response of concrete slabs subjected to large displacements are reported. In particular, the work of Omer [2006] and Herraiz Gómez [2016] was assumed as the basis of the following developments. A new method to assess the response of simply supported reinforced concrete slabs in the membrane stage is then presented in this chapter and a comparison with a numerical tool reported.

Chapter 7 "General conclusions and further research" summarizes the conclusions drawn from each chapter, highlighting the results of Chapters 4, 5 and 6. Besides, future possible research avenues on structural robustness are provided.

Chapter 2

Literature review

2.1 Introduction

In the last decades, important and numerous studies have been conducted on the collapse of structures caused by accidental loss of columns. The interest in studying the effects of extreme loading conditions has been triggered by few catastrophic events occurred in recent years since the Ronan Point Building collapse (London, 1968). These tragic events led many countries to include in their design codes integrity and structural robustness requirements in order to design a structure for accidental events. A rational and well recognized design approach (EN1991-1-7 [2002]) is to provide the structure with the ability to withstand events such as fire, explosions, impact and consequences of human error without being damaged to an extent disproportionate to the original cause. Fulfilment of this requirement identifies a 'robust' structural system.

The accidental column removal is a typical design scenario to consider when checking for robustness. The collapse of columns causes dynamic effects, large deformations of the floor system and high rotations in the beam-to-column connections. The extension of the damage strongly depends on the capability to redistribute loads from the damaged to the undamaged parts and to reach a new stable equilibrium configuration. In this context, the alternate load paths, the ductility offered by the joints and the 3D performance capabilities of the floor system constitute essential factors for a robust structural response.

It has been observed that the load-bearing capacity of reinforced concrete structures under transverse loading can be considerably enhanced as shown in Figure 2.1 if membrane forces in slabs can be developed. In laterally restrained concrete slabs, compressive membrane action develops at moderately low deflections as a result of the horizontal outwards displacements of the slab edges. The slab tries to move outwards, but the horizontal restraints along the edges make it impossible, resulting in the development of

compressive forces as shown in Figure 2.2 (a). The load-bearing capacity of the slab is increased by the M-N interaction (Herraiz Gómez [2016]). After a peak, the curve is in a descending trend due to the concrete softening up to complete crush of concrete. If the slab is reinforced with sufficiently ductile rebars, the increased load can be sustained by the reinforcing bars in tension, and the load-deflection trend is ascending. At larger deflections, the slab tries to move inwards due to the kinematic relationships. The restraints do not allow these lateral horizontal movements, and tensile membrane forces develop along the edges of the slab (Figure 2.2 (b)). These tensile forces increase the load-bearing capacity of the slab through second-order effects created at large deflections. The tensile membrane action continues up to rupture of the reinforcing bars and collapse of the slab. The magnitude of both compressive and tensile membrane actions depends on several aspects, such as the boundary conditions, the concrete compressive strength, the reinforcing proportion, the ductility of the rebars and the slab slenderness (Vessali [2015]).

A large number of studies about progressive collapse assessment is present in literature due to the necessity of enhancing the robustness of structures and include in design codes further and more exhaustive specifications and recommendations. The current chapter begins with a short review of the most relevant catastrophic events in history; then, the main research activity on progressive collapse is presented, focusing on three main directions, i.e. experimental studies, numerical simulations and analytical developments.

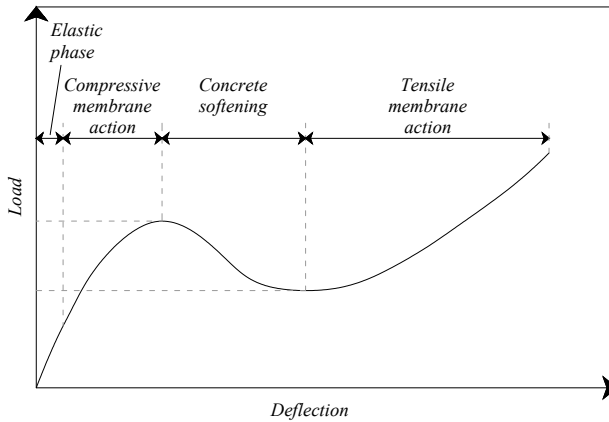


Figure 2.1: Typical load-deflection response of reinforced concrete slabs

2.2 Most relevant events in history

The most relevant and catastrophic events of progressive collapse occurred in the last decades are described in the following. The affected

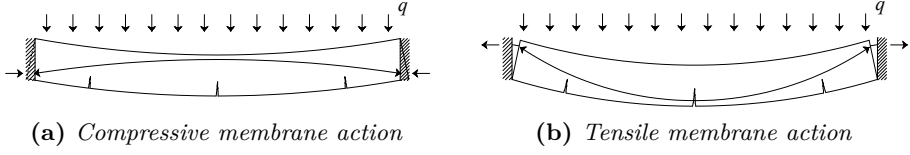


Figure 2.2: *Membrane action in laterally restrained concrete slabs*

buildings suffered partial or total collapse due to the effects of accidental actions, such as explosions, impacts, thermal loads or combinations thereof. These events underline the effects and consequences that limited failures in structures cause in terms of human and economical losses.

World Trade Center

The collapse of the Twin Towers (September 11, 2001) occurred after the impact of two air planes (Figure 2.3), is amongst the worst building disaster worldwide in history and led to the largest human loss from any single building collapse. Even though the aircraft impacts caused important structural damage in the directly affected zone, they did not immediately trigger the total collapse of the towers. The elevated temperature caused by the large amount of jet fuel burning caused the loss of the bearing capacity of the columns in the core of the towers. The dynamic impact of each floor led to the failure of all columns and the complete collapse of the buildings. This event represents a typical example of pancake type collapse mechanism. This catastrophic event is the one with the most large repercussion from all other collapse scenarios worldwide and encouraged researchers studying the progressive collapse scenario and, in particular, structural robustness.



Figure 2.3: *Progressive collapse of the World Trade Center*

Mississippi River Bridge

Several bridge collapses can be cited, but one of the most important one is the I-35W Mississippi River Bridge in Minneapolis (August 1, 2007) were

13 people died (Figure 2.4). The collapse occurred after the rupture of the steel plates joining the structural elements in eight joints of the bridge. This fracture caused the collapse of the central bridge span, leading to collapses all over the bridge length. Inspections before the event did not reveal plastic deformations in the members or other critical signs, meaning that a fracture rupture occurred. The absence of alternative load paths to redistribute loads led the progressive collapse and the complete destruction of the bridge.



Figure 2.4: Progressive collapse of the Mississippi River Bridge

Murrah Federal Building

The partial collapse of the Murrah Federal Building (Figure 2.5) in Oklahoma City (April 19, 1985) was caused by a bomb explosion on the street outside the building. This event claimed 168 lives and more than 680 injured people. The failure of the building was triggered by the local failure of a small part of the structure, possibly due to the collapse of some columns at the base. The propagation of failures caused the propagation of the collapses both horizontally and vertically, resulting in approximately half of the total floor area affected by the event.



Figure 2.5: Progressive collapse of the Murrah Federal Building

Ronan Point

The Ronan Point was an apartment building in London built between 1966 and 1968 that partially collapsed after an explosion (Figure 2.6). The tower of 22 storeys was made up by precast reinforced concrete elements, including the walls, the staircases and the floor slabs. Each floor was directly supported by the walls on the lower stories. The strength of the connections was only ensured by bolts and friction between the elements, providing no redundancy for load redistribution in the event of a local failure. A gas explosion in the 18th floor caused the expulsion of the exterior walls and the total loss of bearing capacity of the part of the building affected by the event. The loss of supports caused the impact of the upper stories below and the progressive collapse.

The Ronan Point brought the progressive collapse phenomenon and structural robustness to the attention of the public, even if it is not the largest building collapse in history. From this event, several studies to enhance structural response for accidental events have been performed, and specific requirements and recommendations have been incorporated in design codes.



Figure 2.6: Progressive collapse of Ronan Point Building

2.3 Progressive collapse resistance of building structures

2.3.1 Experimental studies

Experimental studies about progressive collapse of structures followed several aspects of the research, ranging from small scale joints components to full-scale three-dimensional assemblies. Reinforced concrete structures, steel structures with and without the presence of the slab and composite steel-concrete structures were the object of the studies.

The column loss in a building is one of the most common and effective

damage scenarios recommended for progressive collapse investigation. The collapse of vertical members causes dynamic effects, large deformations of the floor system and large rotations in the beam-to-column connections. In this context, several studies on the performance of steel and composite connections have been recently conducted in order to investigate the axial force-bending moment interaction, and the failure modes and ductility of joints.

Li et al. [2015] investigated the catenary behaviour of welded unreinforced flange-bolted web connections in plane frames by means of full-scale tests and numerical simulations. The results demonstrate that joints were able to sustain important axial force demands developing an effective catenary action via the bolted web following the primary flexural phase. The failure modes of the bolted web varied with different bolt arrangements under the tensile catenary stage. Similarly to Li et al., Oosterhof and Driver [2014] studied the behaviour of shear connections in steel gravity frames under the unique combinations of moment, shear and axial force. The authors conducted 35 full-scale tests on different type of steel shear joints, including welded-bolted single-angle, bolted-bolted single-angle and bolted-bolted double-angle connections. The force and deformation demands that would be expected in the bays adjacent to a removed column is imposed during the tests. The deformation mechanisms, the failure modes of the joints and the load-development history were characterized for each connection. The results showed that, in laterally braced beams, single and double angle connections have similar overall ductility and strength, and that shear-tab and web-angle connections have the ability to resist vertical load following column removal through the development of catenary action. The connection ductility under axial tension is provided by the dominant deformation mechanism of localized plate-bearing effects at bolt locations in shear tabs and welded-bolted angles, and angle unfolding in bolted-bolted angles. Gong [2017] experimentally investigated two types of bolted-angle beam-to-column connections in a double span frame subjected to column removal. The author provided a mathematical model for the test setup based on a component-based spring model for the bolted angles. A new compressive force versus deformation curve was proposed for the angle spring in addition to an existing tensile force-deformation curve. Among the studies about steel bolted-angle joints, Yang and Tan [2013d] performed fourteen tests on bolted angle connections under tension, in which the bolt hole configuration, angle thickness, bolt size, and material properties were investigated. The test results demonstrated that the load increase at large deformation was higher than the yield strength. Furthermore, Yang and Tan proposed a new mechanical model for bolted-angle connections under tension based on the experimental results as shown in Figure 2.7.

In order to study the effect of tensile forces on the response of both semi-rigid and rigid composite joints, Gao et al. [2017] and Guo et al. [2014]

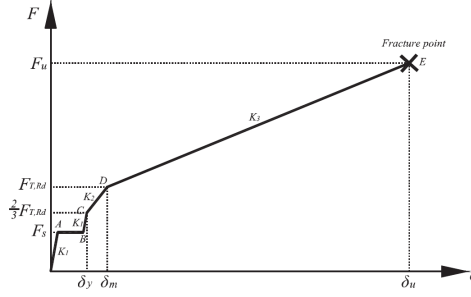


Figure 2.7: Proposed mechanical model of bolted-angle connections under tension (Yang and Tan [2013d])

conducted pure flexural, pure tensile and combined flexural-tensile tests on several flush end-plate and welded connections, respectively. The results highlighted that the semi-rigid composite joints allowed sufficient rotation capacity for the formation of the catenary action in the beams in the case of pure bending moment conditions. Furthermore, the tensile behaviour of the flush end-plate semi-rigid joints is not as good as that of rigid joints. Low strength bolts used in the tests did not influence the initial stiffness of the connections, but reduced the moment and tensile resistance. Similar tests were conducted by Yang and Tan [2013b], who investigated the ductility of composite web cleat and flush end-plate connections in the column loss scenario. The test results demonstrated that, at the initial loading stage, the applied load was resisted by flexural action, while at large deformations, the load was resisted by catenary action. The authors also found that the composite slab could increase the load-carrying capacity of the joints at both flexural and catenary action stages. The collapse mode of the joints (Figure 2.8) shows the fracture of the end-plate and of the angle in the flush end-plate and web-cleat connections, respectively. Studying experimentally different types of bolted steel-beam-column joints under a central column removal scenario was the main object of the research of Yang and Tan [2013c]. The results indicated that the contribution of catenary action to the load resistances of various types of steel joints is important and can be considered to increase the structural capacity of the frames. Furthermore, the rotations of beam-to-column connections at peak loads based on the experimental results were much higher than the recommended values by the US Department of Defense [2009] and ASCE [2006]. Further studies about beam-to-column connections subjected to combined bending moment and axial force due to the collapse of an internal column were conducted by Da Silva et al. [2004] and Weigand and Berman [2015], while Del Savio et al. [2009] developed a generalised mechanical model for semi-rigid connections based on the general principles of the component method, including axial force and bending moment interaction. The method provides a sim-

ple approach to evaluate the bending moment versus rotation curve of the connection for any axial force level.



Figure 2.8: Failure modes of the specimens (Yang and Tan [2013b])

Lew et al. [2012] conducted an experimental study of two full-scale beam-to-column assemblies with two types of moment-resisting connections under a column loss scenario. The connections included a welded unreinforced flange bolted web connection and a reduced beam section connection. Firstly, the assemblies remained in the elastic range at small displacements of the removed column. In this stage, the behaviour of the joints was dominated by flexure. With increased deflections, yielding occurred and axial tension developed in the beams as in the catenary action stage. The failure of the joints tested by the authors was characterized by local buckling of the top flanges of the beams near the removed column, by shear failure of the bolts and by fracture of the bottom flange of the beam near the weld.

Besides research on single members under impact loading and large deformation process, several studies accounted for the influence of the surrounding structure by means of experimental tests on two-dimensional frames. Most of the studies about 2D assemblies comprised of three-column and two-beam frames subjected to the central column loss simulated by means of a hydraulic actuator. Sadek et al. [2011] presented an experimental and computational study of the performance of reinforced concrete and steel beam-column frame under the monotonic vertical displacement of the central column. The steel frame failed by shear fracture of the bolts in the joints connecting the beam with the central column and by the fracture of the bottom beam flange near the weld. The concrete assembly was characterized by excessive crack openings in the concrete in correspondence of the supports and by rupture of the steel reinforcement in tension as shown in Figure 2.9. A reinforced concrete frame under middle column removal scenario was the object of the research of Yu and Tan [2012]. The authors statically investigated the behaviour of six beam-column subassemblies in order to evaluate the developing of alternate load paths to mitigate progressive collapse. With adequate lateral axial restraints, both compressive

arching stage and tensile catenary stage can be mobilised enhancing significantly the structural capacity. Similarly to the tests by Sadek et al. [2011], the specimens failed by the rupture of the steel reinforcing bars in the bottom and top layers of the beam near the attachment with the central and lateral columns, respectively. Similar results were obtained by Ren et al. [2016] and Yu and Tan [2013], who tested 2D reinforced concrete frames subjected to column loss.

Demonceau and Jaspart [2010] tested a 2-dimensional composite steel and concrete frame for a column loss scenario. Four beams and five columns composed the frame object of the study. The lateral beams were axially restrained in order to allow developing the catenary action at large deflections. In the first phase of the test, the gravity load on the slab was applied by means of concrete blocks; furthermore, the column removal was simulated by means of a hydraulic jack replacing the central column. The load-deflection curve of the removed column, reported in Figure 2.10, indicates that large axial forces can develop in the beam if the frame is laterally restrained. The high ductility of the frame, confirmed by the joint rotation results, is significant to allow a large vertical displacement of the central column. The observed frame at collapse, pictured in Figure 2.11, shows full-depth cracks in the concrete in correspondence of the hogging moment region in the lateral supports, significant shear deformation of the column web in the lateral joints, and important plastic deformations of the end-plate in the central joints. A similar test on a composite steel and concrete frame was conducted by Yang et al. [2016] and Guo et al. [2013]. At collapse, the bolts in the bottom row of the flush end-plate joints connecting the beam with the removed column fractured, and severe buckling of the beam bottom flange near the lateral column occurred.



Figure 2.9: Failure modes of the steel and concrete assemblies (Sadek et al. [2011])

Focusing on steel framing systems, Dinu et al. [2017] and Xu et al. [2018] experimentally and numerically tested 2D subassemblies under the internal column loss scenario. Four different types of beam-to-column joints were considered in the tests; a welded cover plate flange joint, a haunch end-plate bolted joint, a reduced beam section welded joint, and an unstiffened extended end-plate bolted joint. The results indicated that the first three

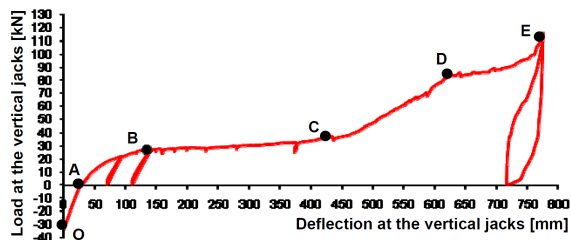


Figure 2.10: Load-deflection curve obtained from the test of (Demonceau and Jaspart [2010])



Figure 2.11: The frame tested by (Demonceau and Jaspart [2010]) at collapse)

types of joints exhibited good ductility, with the catenary action that significantly increased the ultimate load resistance of the specimens. The steel frame with the extended end-plate connections showed the lowest ductility and ultimate load resistance, due to the tensile fracture of the bolts in the bottom row of the joint, preventing the formation of the catenary action in the beams. For this purpose, the authors suggested that bolt strengthening is necessary for improving the collapse resistance of the structures.

Studying 2D subassemblies subjected to large displacements allows the evaluation of the behaviour of beam-to-column connections and the influence of the compressive arching action and catenary action developed in the beams on the overall structural response. 2D effects, such as membrane action in concrete slab, cannot be evaluated by these type of experiments. To date, it is well known that the flooring system and the slab play a key role in the progressive collapse of buildings and substantially contribute to increase the load resistance and the robustness of the structure (El-Tawil et al. [2013]). For this purpose, several authors studied the response of isolated concrete slabs subjected to large displacements.

Bailey and Toh [2007] and Foster et al. [2004] experimentally tested small-scale horizontally unrestrained concrete slabs at ambient and elevated temperatures subjected to large vertical deflections. The tests aimed to investigate the influence of the reinforcement on the load capacity of the slab by the mobilisation of tensile membrane action. The results showed that

the strains are located along the diagonals of the slab, and the slab panels bounded by the yield lines rotate rigidly about these lines, confirming the assumptions of the 'yield line theory' (Hognestad [1953]). Both tests at ambient and elevated temperature showed that the slabs failed by fracture of the reinforcing bars at the centre of the slab, with vertical displacements ranging from span/12 to span/6.

Qian and Li [2017] conducted a series of dynamic tests to study the dynamic behaviour of beam-column-slab substructures subjected to the sudden loss of an edge column. Furthermore, the residual strength behaviour of the specimens was investigated by conducting subsequent push-down tests applying a load onto the slab. The results indicated that the damage caused by the dynamic behaviour significantly decrease the initial stiffness and the efficiency of the compressive arching and membrane actions, while the load resisting capacity is derived mainly from the tensile catenary and membrane actions in large deformation stage. The dynamic behaviour of structures was also investigated by Qian and Li [2013], who tested two series of reinforced concrete beam-column substructures with and without the slab subjected to the sudden removal of the corner support. The comparison of the results between the two series of specimens allowed concluding that the slab significantly increased the yield strength and initial stiffness of the specimen, and the ductility at large deformations.

The static behaviour of a reinforced concrete slab after the loss of a corner column was studied by Kai and Li [2012]. The authors also investigate the evolution of the cracking pattern at different performance levels of the slab (Figure 2.12), and demonstrated that the slab increased the ultimate resistance capacity of the structure by up to 63.0%, significantly reducing the likelihood of progressive collapse with respect to the outcomes of Qian and Li [2012].

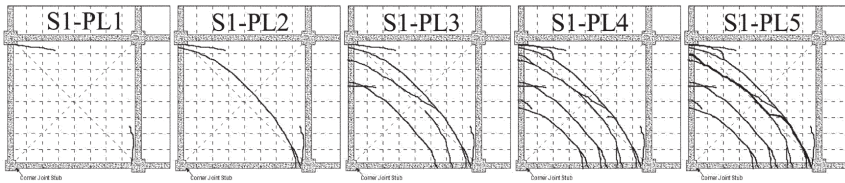


Figure 2.12: Observed cracking patterns of the slab at different performance levels (Kai and Li [2012])

The behaviour of reinforced concrete beam-slab substructures subjected to a penultimate-external column removal was studied by Lim et al. [2017], Dat and Tan [2014b] and Lu et al. [2017b]. In this damage scenario, the observed cracking pattern in the slab, illustrated in Figure 2.13, shows that in the top side of the slab, the cracks revealed a sort of semi-concentric rings

around the removed column, while in the bottom side the cracks are oriented from the slab corner to the interior column along diagonal directions. The authors demonstrated that the structural capacity of the beam-slab specimens under the beam mechanism and the catenary mechanism was increased by 146% and 98%, respectively, compared to those of the beam specimens. Similarly to Kai and Li [2012], Qian et al. [2016] studied the influence of multicolumn removal on the behaviour of reinforced concrete floors. The authors asserted that edge and corner column removal has the effect of decreasing the ultimate capacity of the specimen. This large decrease was due not to the increase of the span, but to the fact that any beneficial effect, such as compressive arching action, tensile catenary action, compressive and tensile membrane actions, can not be activated. In the case of internal column removal (Dat and Tan [2013] and Qian et al. [2014]) full compressive and tensile membrane actions can be developed in the concrete slab. The tensile membrane action began to be activated at 4.5-5.% of the beam span, which is much earlier than the onset of catenary action (10% of beam span) (Qian et al. [2014]).

Even if some experimental campaigns were carried out, the current literature focuses mainly on the 2D behaviour of frames or on the isolated behaviour of the slab by itself. In order to improve the knowledge of the actual frame response, experimental tests on 3D framed structures are the most complete and recommended approach (Zandonini et al.). These experiments are expensive and complex, and, to date, few experimental data are available.

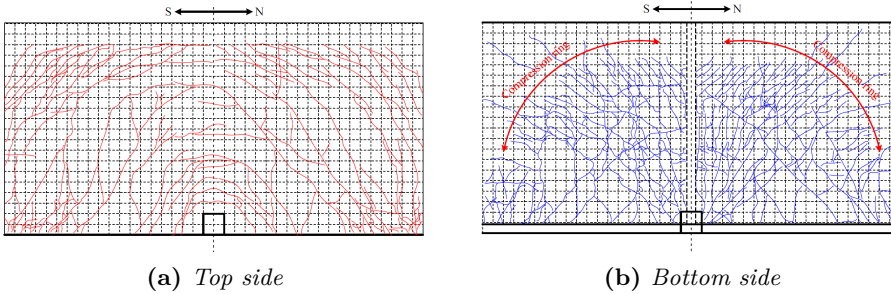


Figure 2.13: Cracking pattern in the slab (Lu et al. [2017b])

Dinu et al. [2016] and Li et al. [2017] experimentally and numerically tested 3D steel frame systems under the central column loss scenario. The specimens were extracted from the full reference structures and tested statically by applying a force by an hydraulic actuator at the removed column. The force-deflection relationship, illustrated in Figure 2.14, shows a good structural behaviour of the specimen, and the beam-to-column connections had sufficient strength and ductility to resist the catenary forces developed

in the beams.

The response of a composite steel-concrete 3D frame was investigated by Fu et al. [2017], who conducted an experimental test on a scaled-down specimen extracted from the prototype structure subjected to the loss of the internal column. The composite slab consisted of a profiled steel decking and a concrete slab, while the beam-to-column connections were bolted flush end-plate and bolted double angle-cleat. Even if the floor system consisted of a composite slab, tensile membrane action developed in the slab; the beam-to-column joints revealed sufficient deformation capacity, allowing the failure modes of the composite floor system to be developed in a ductile way. Further experimental analysis on 3D composite steel and concrete framed structures were conducted by Johnson et al. [2015] and Jahromi et al. [2012].

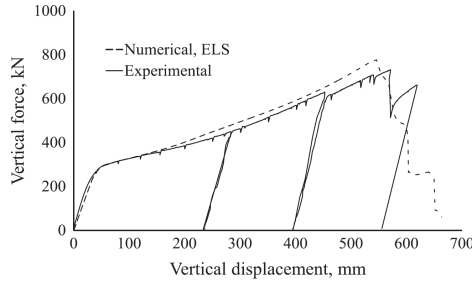


Figure 2.14: Force-deflection relationship of the 3D steel framed structures (Dinu et al. [2016])

2.3.2 Numerical studies

Considering finite element analyses, Guo et al. [2015] numerically studied the structural performance of semi-rigid composite frames under the column loss scenario. The two-dimensional frame was assembled with four beams supported by five columns, and with flush end-plate beam-to-column connections with two rows of bolts. The FE model was developed using both 3D solid elements for the structural members close to the removed column, and 2D shell elements and 1D beam elements for the other regions of the frame. The results showed that the progressive collapse resistance of the composite frame is sensitive to the properties of the bolts. In particular, increasing the fracture strain of the bolts improves significantly the progressive collapse resistance, and increasing the diameter of the bolts increases the load-carrying capacity and ductility of the connections. Yang and Tan [2012] numerically studied six types of beam-to-column connections: web cleat, fin-plate, top and seat with web angles, flush end-plate, and extended end-plate. Both static and dynamic analyses were conducted in order to overcome convergence problems. The results indicated that the static solver

could predict with more accuracy the behaviour of the connections than the explicit dynamic solver. But the complete fracture simulations could only be conducted by explicit dynamic analyses, although huge computation resources were required.

The dynamic response of multi-storey steel composite buildings subjected to column removal was investigated by Fu [2009] and Fu [2012] by means of non-linear dynamic FE analyses. The structural members were modelled with shell and beam elements incorporating both non-linear material characteristics and non-linear geometric behaviour. The analyses indicated that all the structural members at the possible column removal level should be designed to have the axial capacity of at least twice the static axial force of the members, and that a column removal at a higher level will induce larger deflections than a column removal at ground level. Furthermore, as the slab cracks are concentrated in correspondence of the removed column area, the author suggested to increase the steel reinforcement in the concrete slab to help prevent progressive collapse. Further non-linear dynamic simulations of progressive collapse for a multi-storey building were conducted by Kwasniewski [2010], showing that detailed modelling can provide a realistic approximation of the structural collapse. Li and El-Tawil [2013b] numerically investigated using 3D non-linear models the robustness of a seismically designed steel moment frame building. The studies were conducted to gain insight into the role of the composite action between the concrete slab and the steel beams and the slab membrane action in structures subjected to column loss. The results suggested that the building can be more vulnerable to loss of columns in the upper stories than in the lower ones, and that the slab can contribute significantly to the robustness of the structure.

The behaviour of the floor slab was studied by several authors, with particular focus on the compressive and tensile membrane actions developed at large displacements. Yu et al. [2018b] numerically studied the progressive collapse resistance of reinforced concrete beam-slab substructures under a perimeter column removal scenario. The simulation results showed that the collapse resistance is developed primarily through the compressive arching action of the longitudinal beams and the flexural mechanism of the transverse beam at small deflections, and through catenary action of the beams as well as tensile membrane action of the concrete slab at large deflections. The effect of the compressive and tensile membrane actions in beam-slab systems under progressive collapse was also investigated by Pham et al. [2017b], Genikomsou and Polak [2017] and Keyvani et al. [2014]. Different loading configurations and boundary conditions were also studied by the authors. Yu et al. [2010a] conducted FE analysis and parametric studies to evaluate the influence of the joints and of the concrete slab on the effective tying of steel beams. Methods to prevent the progressive collapse of the structure that can be considered in design were proposed. Alashker et al. [2010] presented a paper discussing the progressive collapse resistance

of steel-concrete composite floors in which the steel beams are attached to the columns through shear tabs. Key parameters influencing the robustness of composite floors subjected to column loss were investigated, such as the deck thickness, the steel reinforcement and the number of bolts in the shear tab connections. The results showed that the collapse resistance comes from the steel deck, and that the increase of the connection strength is not beneficial to the increase of the overall collapse strength. Further finite element analyses on the effects of column removal in composite floor systems and reinforced concrete subassemblies were developed by Main [2014] and Bao et al. [2012].

2.3.3 Analytical development for progressive collapse assessment

Stylianidis and Nethercot [2015] developed a mechanical approach for evaluating the behaviour of steel and composite steel and concrete connections for use in progressive collapse analyses considering the interaction between bending moments and axial load. The connection performance is one of the most important feature of the structural response under the column removal scenario. Besides, an accurate modelling of the joints behaviour is an important component for any realistic analyses. In addition to the connections behaviour, further analytical methods for the prediction of the floor system response have been developed in recent years. Burgess [2017] provided an analytical approach to quantify the tensile membrane action in lightly-reinforced thin concrete slabs at large deflections. The tensile membrane action is used to enhance the load carrying capacity of a structure, both at ambient and elevated temperature. The author modified the basic assumptions of the traditional yield-line theory in order respect the equilibrium and the kinematic assumptions of the method. Herraiz and Vogel [2016] proposed a novel design approach for the analysis of laterally unrestrained reinforced concrete slabs considering membrane action. The full load-deflection response of the slab from the undeformed configuration to the collapse condition can be computed by means of a kinematic model. Similarly to Herraiz and Vogel [2016], Bailey [2001] presented an analytical method based on an equilibrium approach for the evaluation of the membrane action in unrestrained concrete slabs. An experimental campaign was conducted and the results compared with the analytical ones showing a good agreement. Further contributions to the study of concrete slabs subjected to large displacements have been developed by Wang et al. [2018], Alashker and El-Tawil [2011] and Kim et al. [2015].

Dat et al. [2015] presented a simplified approach for the progressive collapse assessment of reinforced concrete building structures subjected to a column removal. The collapse resistance is based on the elastic-plastic response of a double-span beam-slab substructure, in which the yield-line theory allows

computing the ultimate flexural capacity of the system, and the curvature ductility of the connections in the affected area governed the displacement ductility at the removed column position. Further analytical methods to evaluate the progressive collapse of steel frame structures subjected to column loss are developed by Chen et al. [2016], Gerasimidis [2014] and Kim and Kim [2009].

Izzuddin et al. [2008] developed a simplified framework for the assessment of structural performance following sudden column loss. The method can model the dynamic features of the problem without conducting a detailed dynamic numerical analysis, and allows application at various levels of structural idealisation, such as members, single-floor system, multiple-floor system, affected bay of entire building structure, as shown in Figure 2.15. The response at higher level is assembled by the responses at lower levels basing the method on a simplified multi-level approach. Three main stages are employed in the method to obtain the pseudo-static response of the system: i) the determination of the non-linear static response, ii) the dynamic assessment using an energy balance method, iii) and a ductility assessment of the beam-to-column connections. The authors identified the pseudo-static capacity of the system as a new and rational measure of building robustness under sudden column removal scenarios. The approach is illustrated in Figure 2.16 where, for two levels of applied gravity loads ($\lambda_1 P_0$ and $\lambda_2 P_0$), the maximum dynamic displacements ($w_{d,1}$ and $w_{d,2}$) are calculated from the non-linear static response based on the equivalence of the two hatched areas.

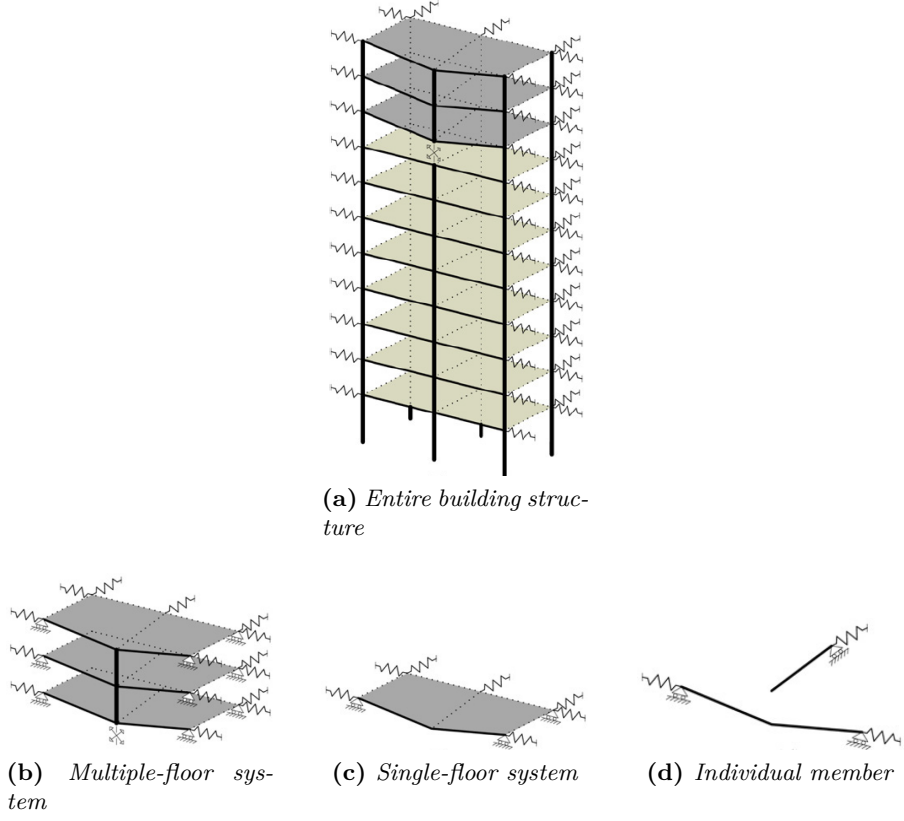


Figure 2.15: Simplified multi-level approach (Izzuddin et al. [2008])

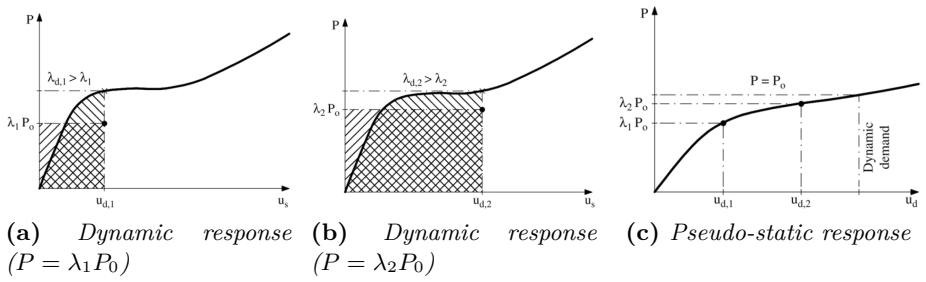


Figure 2.16: Simplified dynamic assessment (Izzuddin et al. [2008])

Chapter 3

Robust design of building structures

3.1 Introduction

Since the collapse of Ronan Point Building in 1968, some codes and standards have included some specifications to avoid the progressive collapse of building structures. In this chapter the main European Codes on the design of structures against accidental events are described .

3.2 Guidelines and design codes

3.2.1 British Standards

The British Standards 5950-1:2000 [2001], strongly motivated by the event of Ronan Point in UK (see Figure 2.6), were the first Codes in Europe to include the problem of the structural robustness. According to the British Standards, the building structure should be constructed so that, in the event of an accident, it will not suffer collapse to an extent disproportionate to the cause. In order to reduce the risk of localized damage spreading, the buildings should be able to sustain substantial permanent deformation of members and connections. Three main strategies are proposed in order to ensure a minimum level of strength to resist accidental events to the structures. First of all, the buildings should be effectively tied together at each principal floor level. Each column should be effectively held in position by means of horizontal ties in two directions as illustrated in Figure 3.1. The horizontal ties may be steel members, steel reinforcing bars embedded in the concrete slab and anchored to the steel members and steel mesh reinforcement in a composite slab with profiled steel sheeting. All the horizontal ties should be capable of resisting a factored tensile load of not less than 75 kN. Furthermore, if the resistance of horizontal ties

is not sufficient to ensure a robust response, it is recommended that the structure should be designed to sustain a member removal such a column, and that the area of collapse should be limited and localised. The portion of the building at risk of collapse should not exceed 15% of the floor or roof area or 70 m^2 at the relevant level and at one immediately adjoining floor or roof level. If the notional removal of a column, or of an element of a system providing resistance to horizontal forces, would risk the collapse of a greater area, that column or element should be designed as a key element capable of sustaining additional loads (5950-1:2000 [2001]).

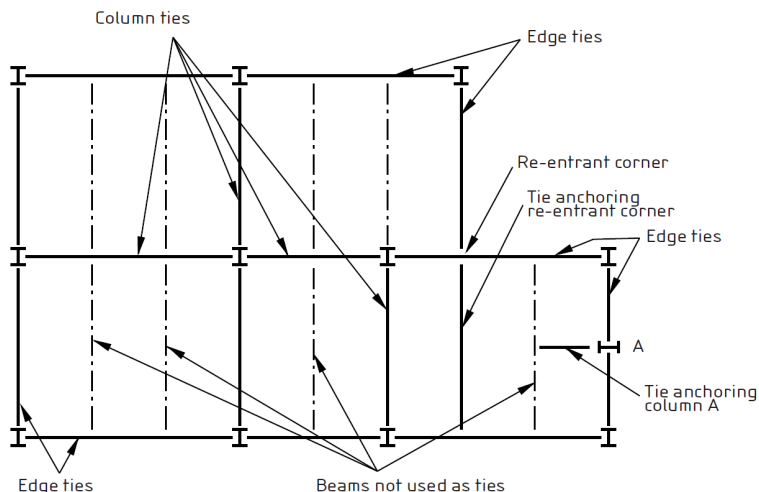


Figure 3.1: Example of tying the columns of a building (5950-1:2000 [2001])

3.2.2 Eurocodes

The approach to robustness adopted by EN1991-1-7 [2002] is based on the design criteria provided by EN1990 [2002] which, similarly to 5950-1:2000 [2001], states that a structure shall be designed and executed in such a way that it will not be damaged by events such as explosions, impact, and consequences of human errors to an extent disproportionate to the original cause. Firstly, it is important to establish if or when the structure shall be designed against accidental loadings. For this purpose, EN1991-1-7 [2002] provides three classes of consequences, depending on the impact that the collapse of the structure would produce on society, evaluated by considering the consequences on people, on the environment, and on the economy. These three classes are:

- CC1: low consequence of failure. No specific consideration is necessary for accidental actions except to ensure that the robustness and stability rules given in the Eurocodes are followed;

- CC2: medium consequence of failure. Depending upon the specific circumstances of the structure, a simplified analysis by static equivalent action models may be adopted or prescriptive design/detailing rules may be applied;
- CC3: high consequence of failure. An examination of the specific case should be carried out to determine the level of reliability and the depth of structural analyses required. This may require a risk analysis to be carried out and the use of refined methods such as dynamic analyses, non-linear models and interaction between load and structure.

EN1991-1-7 [2002] distinguishes the accidental actions in identified and unidentified actions as illustrated in Figure 3.2. In case of unidentified accidental actions, it is important to provide the structure with a sufficient robustness to survive for a period sufficient to facilitate the safe evacuation and rescue of the people from the building. To this aim, EN1991-1-7 [2002] provides several design strategies to be adopted:

- design of key elements with respect to an unidentified accidental action as in 5950-1:2000 [2001];
- structural design so that a localised damage does not cause a spread of the damage in other parts of the structure;
- adoption of prescriptive design rules providing a sufficient robustness to the structure.

These general concepts are specialised by considering the classes of consequences as:

- CC1: structural design following the criteria prescribed by the Eurocodes 0-9;
- CC2a: design strategies as for CC1 and use of horizontal ties in framed structures as shown in Figure 3.3 or anchorages of suspended floor to walls for load-bearing wall construction;
- CC2b: design strategies as for CC1, use of horizontal ties together with vertical ties in all the supporting columns and walls (Figure 3.3). Alternatively to the last provisions, in case of the removal of each supporting column and each beam supporting a column or any nominal section of load-bearing wall, the structural stability should not be compromised and any local damage does not exceed a fixed limit as illustrated in Figure 3.4, where A is the local damage not greater than 15% of the floor area and B is the removed column;

- CC3: systematic risk analyses of the structure should be performed taking into account foreseeable and unforeseeable hazards. The design strategies to be adopted could be structural (structural design against suitable action levels, alternative load paths) or non-structural such as reduction of the occurrence of the action, limiting the intensity of the events and of the related consequences.

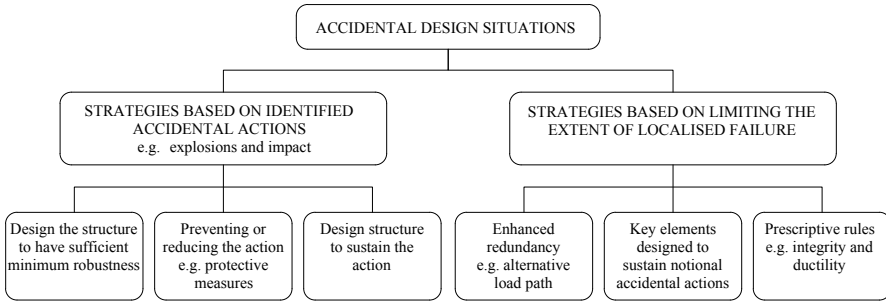


Figure 3.2: Possible strategies for accidental design situations (EN1991-1-7 [2002])

Design criteria to guarantee a minimum level of robustness to the structure (e.g. alternate load path) are provided only for consequence classes 2a and 2b. For consequence class 3, the Eurocode 1 EN1991-1-7 [2002] indicates possible strategies to prevent the progressive collapse of the building to be specialized to the particular scenarios. For this class, in which a risk analysis is prescribed, the adoption of preventative measures is dependent on the adoption of an acceptable risk level. In the case where prevention measures are needed, the possible strategies to adopt are:

- structural or non-structural measures to mitigate the event;
- control the evolution of the event;
- reduce the magnitude of the damage by controlled failures;
- design structures to guarantee sufficient reserve of strength, ductility, robustness and redundancy.

3.3 Concluding remarks

In this chapter, a brief description of the main recommendations provided by the principal European design codes is given. The methods proposed by the Eurocode 1 for robust design EN1991-1-7 [2002] are widely inspired by the British Standards 5950-1:2000 [2001]. Three principal methods to avoid the progressive collapse of buildings are described: the tying

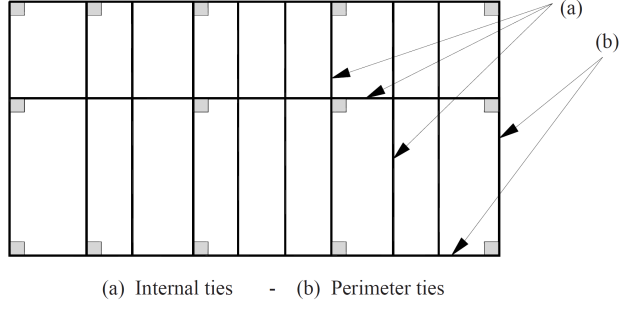


Figure 3.3: Horizontal ties to be adopted in building structures

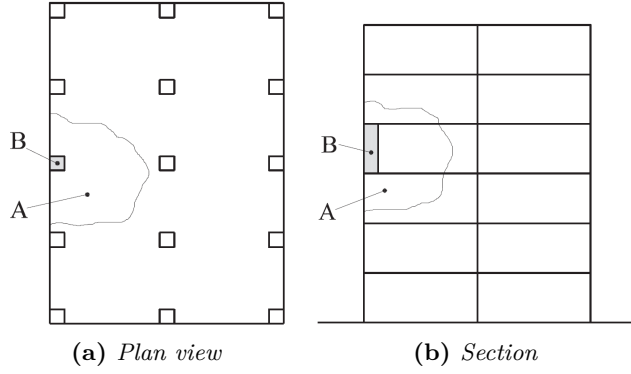


Figure 3.4: Limit to the damage area (EN1991-1-7 [2002])

method, the bridging method and the key element method. The choice strategies to adopt depends on the accidental actions that can be identified or unidentified. For the first ones, possible strategies are to design the structure to have a sufficient robustness, to prevent or reduce the action or to design the structure to sustain the action. For the second ones, it is suggested to enhance the redundancy, to design key elements to sustain notional accidental actions or to follow prescriptive rules to ensure ductility and integrity.

The actual design codes do not provide specific recommendations to design structures for accidental actions and do not identify the domains for which additional investigations are required (Demonceau [2008]), underlining the fact that a significant revision of the recommendations is needed.

Chapter 4

Building structures subjected to column loss

4.1 Introduction

The column removal scenario in buildings is one of the most common and typical design scenarios to consider for checking the robustness of structures. The collapse of columns causes dynamic effects, large deformations of the floor system, and large rotations in the beam-to-column connections. The extent of the associated damage strongly depends on the capability to redistribute loads from the damaged to the undamaged parts and to reach a new stable equilibrium configuration. In this context, the alternate load paths, the ductility provided by the joints, and the 3D performance capability of the floor system constitute essential factors for a robust structural response.

This chapter presents an experimental study conducted at the University of Trento and numerical analyses on 3D steel and concrete composite frames subjected to a column loss. The sub-frames tested and modelled were extracted from the full-frame reference buildings object of the study. The finite element model was calibrated against the experimental results and assumed as reference model to study the response of the full-frame structures. The comparison of the structural response between the sub-frames and the full-frames is also reported and commented.

4.2 The experimental programme

The experimental tests were conducted at the University of Trento among the activities of the European Research Project RFCS 'RobustImpact' European Commission [2012]. The project aimed at developing new design concepts for steel and concrete composite structures against accidental events.

The study focused on the key aspects of the performance of composite structures, and included a large number of static and dynamic tests on individual members and sub-structures.

Two full-scale 3D steel and concrete composite sub-frames with solid slab and end-plate beam-to-column connections were tested simulating the total loss of a column. The tests enable investigation of the redundancy and alternate load paths provided by the floor system in terms of activation of different resistance mechanisms including slab membrane effects. Equivalence between the sub-frame and the full-frame required a series of careful numerical analyses, which enable the definition of the boundary restraints of the specimens and of the loading sequence in the tests as described in Zandonini et al..

4.2.1 The case study frames

The experimental work in the 'RobustImpact' project was associated with two frames referred to as case studies. Two five-storey composite steel and concrete office buildings were selected representing a popular solution. In particular, two different geometric configurations of the frame were considered: the first configuration is symmetric with respect to both in plane directions, while the second one is symmetric only with respect to one in plane direction. The two frames will be referred to hereinafter as '*symmetric*' and '*asymmetric*', respectively. The total dimensions of the buildings are 34.2 m in the X direction (six bays) and 11.4 m in the Y direction (two bays), while the total height is 18.0 m. Figure 4.1 and Figure 4.2 illustrate the plan and front views of the reference structures.

The position of the steel braces designed to resist the horizontal actions in the Y direction is asymmetric in both frames (see Figure 4.1). This solution, despite being less effective with respect to the seismic performance, has been adopted in order to avoid the presence of the steel braces in the sub-frames to be experimentally investigated. This makes the sub-structures more representative of a general case.

The design was based on the Eurocodes, and no seismic considerations were made in order to decouple the issues of seismic and robust design. Structural design aimed at obtaining for both structures the same sections of steel structural elements: HEB220 for the columns and IPE240 for the beams, thickness of the concrete slab is taken as 150 mm (nominal value). Beam-to-column joints (Figure 4.3), the same in both structures, were bolted flush-end-plates designed according to the component method as in the Eurocode EN1993-1-8 [2005]. This criterion was chosen in order to reduce the number of variables and to simplify the comparison of results between the two structures. The rebar layout in the concrete slab is hence the only nominal difference between the frames. The rebar cover in the concrete slab is

20 mm. A welded wire mesh of $\phi 10/150 \times 150$ mm was located at the top and bottom side of the slab, while additional reinforcing bars were required in several zones. Full shear connection between steel beams and concrete slab was reproduced with shear studs. Structural steel grade S355, rebars grade B450C, bolts class 10.9 and concrete C30/37 were selected for the materials of the structural elements. More details about the design of the reference structures can be found in Baldassino et al. [2018].

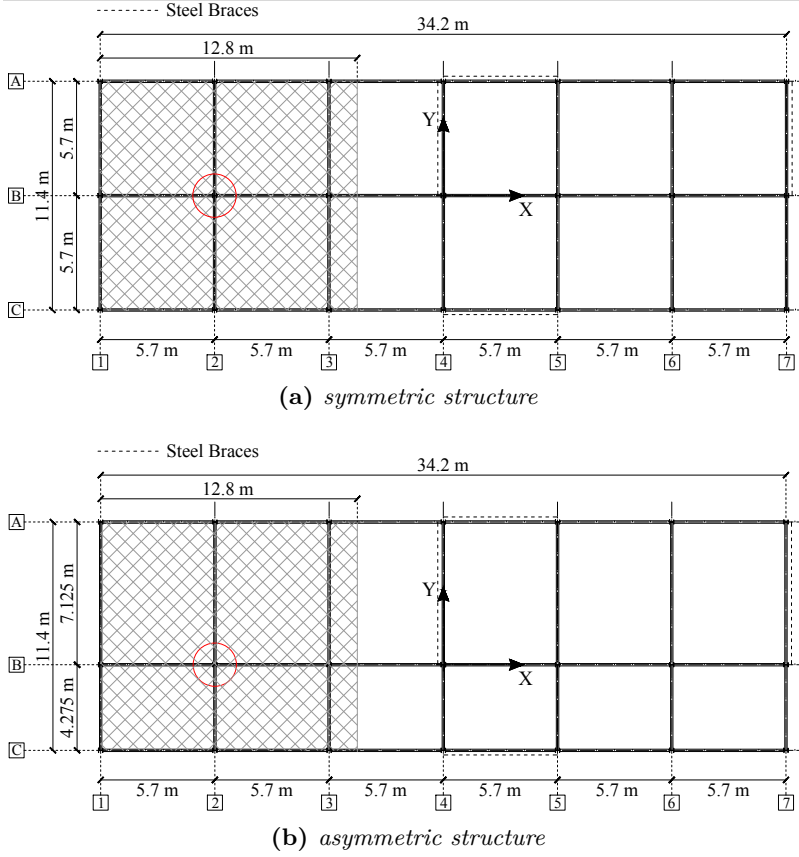


Figure 4.1: Plan views of the reference full-frame structures

4.2.2 The full-scale tests

The tests were performed on full-scale sub-frames extracted from the reference buildings. The goal was to investigate the frame performance under the column removal scenario.

The sub-frames

The 2×2 bays sub-frames were extracted from the first floor of the corresponding reference building, as illustrated by the dotted area in Figure 4.1.

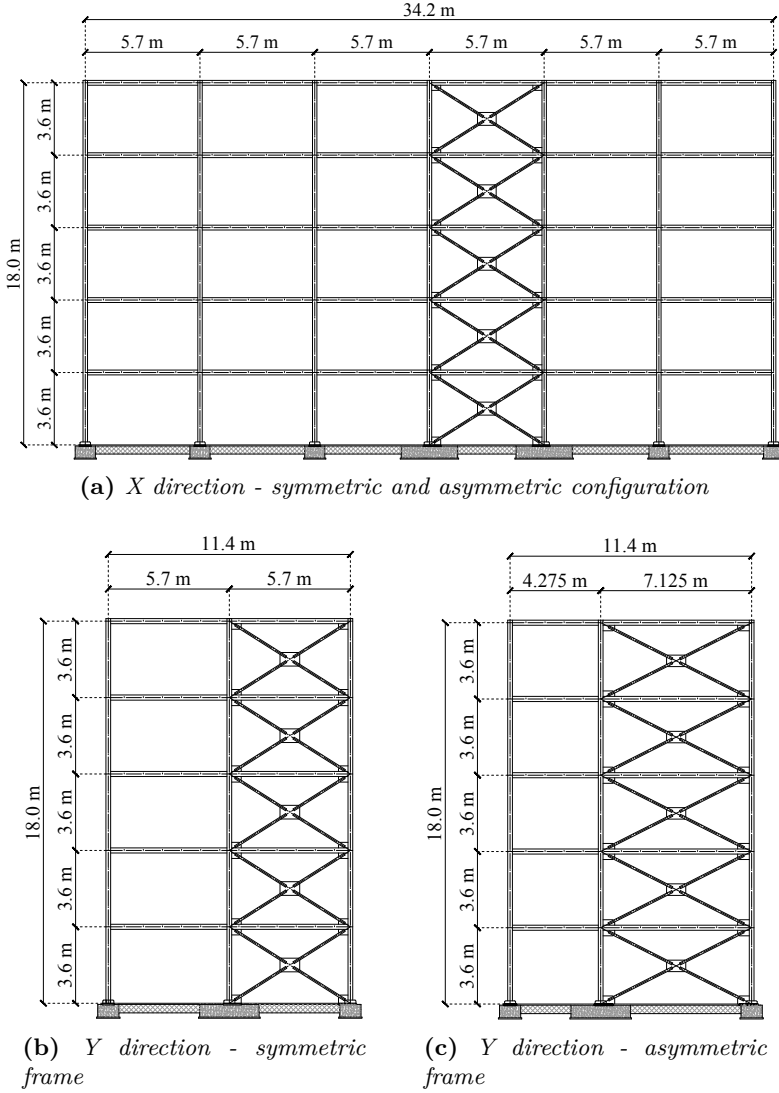


Figure 4.2: Elevation views of the reference full-frame structures

The red circle highlights the column that was removed during the test. More details about the sub-frames can be found in Zandonini et al.. The plan views and plane sections of the *symmetric* and *asymmetric* sub-frames are illustrated in Figure 4.4, while the test set-up is shown in Figure 4.5. The figure illustrates as well the restraining system, which was studied and designed in order to minimize the behavioural differences between the full-frames and the sub-frames. A system of crowning truss members pinned to the top end of the columns takes into account the influence of the upper stories present in the reference structures. The length of the columns above the flooring system approximates the position of the contra-flexure points

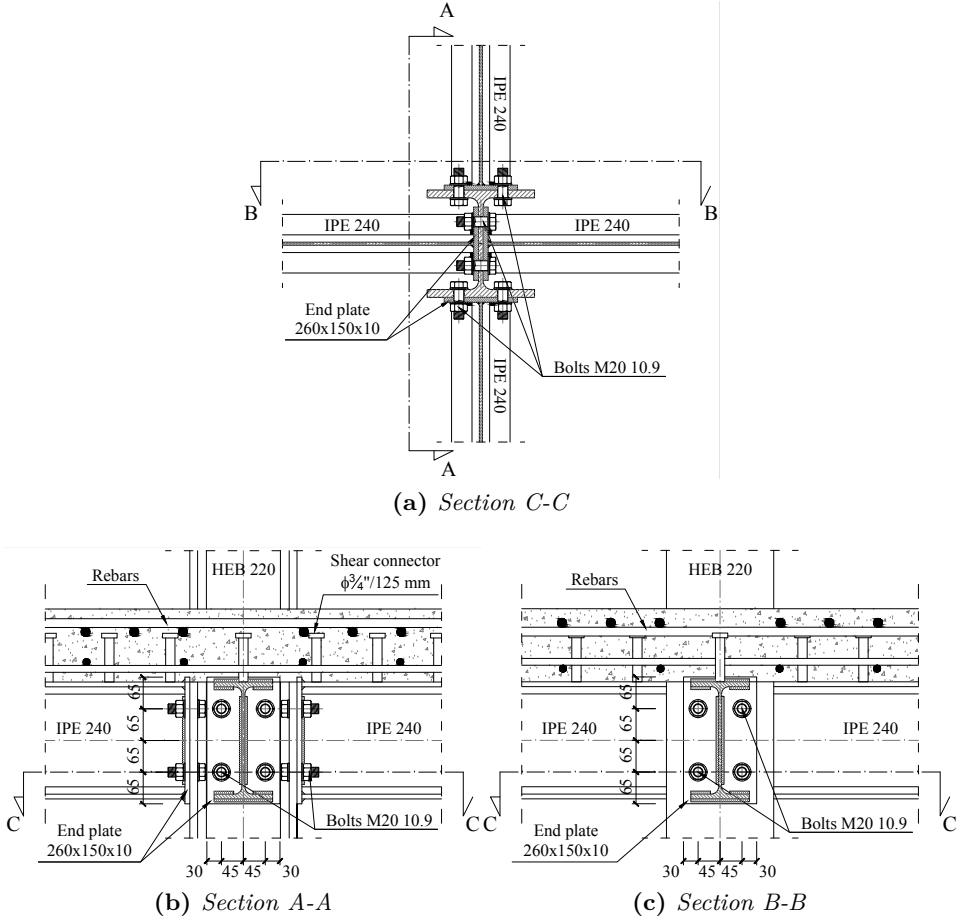


Figure 4.3: Beam-to-column connections (units in mm)

in the second storey.

The specimens were built and tested inside the Laboratory of Materials and Structures Testing of the University of Trento. The construction of the frames started with the erection of the steel skeleton connecting the columns with the strong floor of the laboratory and the lateral bracings to the reaction-walls. Afterwards, the formwork was put in place, the reinforcement positioned and the concrete poured. The central column was 'replaced' by a hydraulic actuator in order to simulate the column loss.

A check about the discrepancies between the design geometry and the real structure was then conducted. In particular, the average value measured for the slab thickness was of 165 mm, 15 mm higher than the design value of 150 mm. Such a difference is due to the complexity of the rebar positioning.

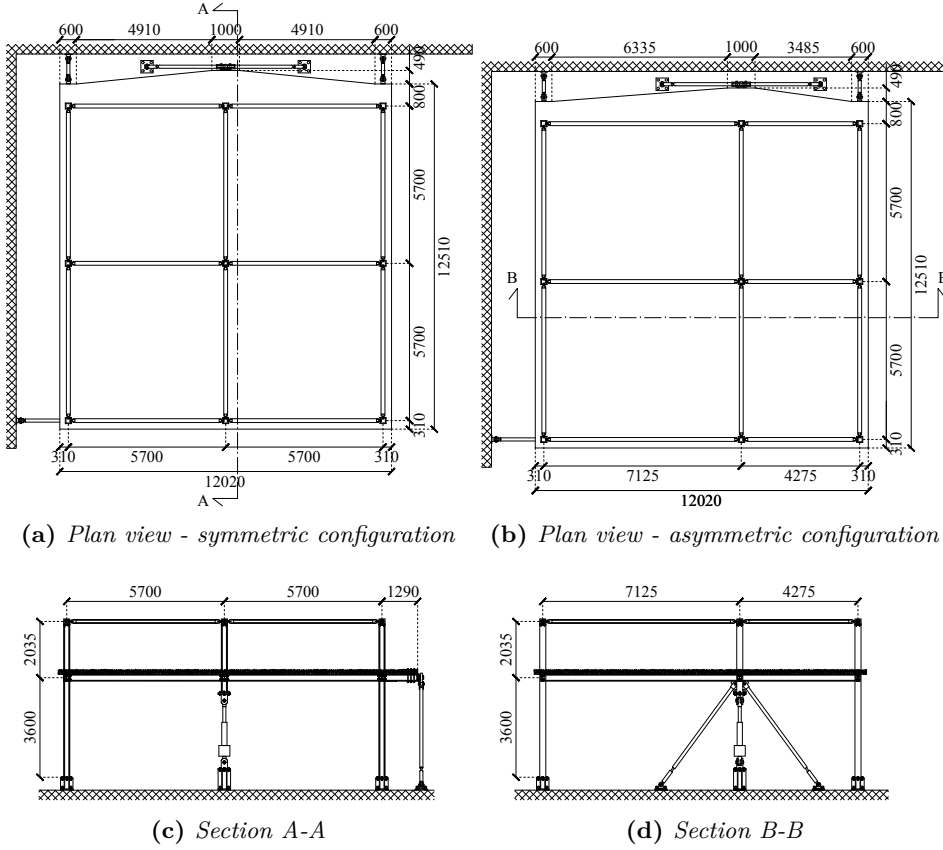


Figure 4.4: Actual dimensions of the sub-frames (units in mm)

During the frames construction, material samples were prepared to perform tests for the characterization of the mechanical properties of the materials (Zandonini et al.). The properties are summarized in Table 4.1 and Table 4.2 for the reinforcement and structural steel, respectively, and in Table 4.3 and Table 4.4 for the concrete.

Table 4.1: Mechanical properties of the reinforcement bars

Rebars diameter (mm)	f_y (MPa)	f_u (MPa)
10	496	586
16	523	631

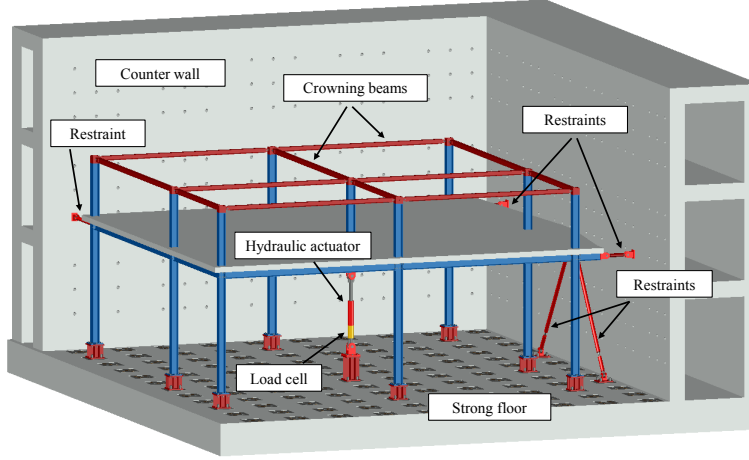


Figure 4.5: The test set-up - asymmetric configuration

Table 4.2: Mechanical properties of the structural steel

Component	f_y (MPa)	Average f_y (MPa)	f_u (MPa)	Average f_u (MPa)	A (%)
Column HEB220	300	303.3	441	440.3	34.9
	306		442		34.5
	304		439		36.1
Beam IPE240	383	409.3	537	540.7	28.2
	391		541		27.0
	454		544		33.3

Table 4.3: Concrete properties - symmetric specimen

Type of test	Concrete age (days)	Curing conditions	R_{cm} (MPa)	f_{ctm} (MPa)
Compression tests	7	Air	43.83	-
	28	Air	56.47	-
	102	Air	65.74	-
Splitting tests	28	Air	-	3.81
	102	Air	-	4.25

The specimens were extensively instrumented in order to maximize the information gained from the tests. Due to the complexity of the frame response, the key parameters to be measured were first identified. The instrumentation set-up is illustrated in Figure 4.6. The attention was focused on the response of the columns, beams, joints and on the behaviour of the slab. More details about the instruments are described in the paper by Zandonini et al..

Table 4.4: Concrete properties - asymmetric specimen

Type of test	Concrete age (days)	Curing conditions	R_{cm} (MPa)	f_{ctm} (MPa)
Compression tests	8	EN 12390-2	45.91	-
		Air	42.47	-
	33	EN 12390-2	54.48	-
		Air	53.69	-
	111	EN 12390-2	57.59	-
		Air	57.34	-
Splitting tests	33	EN 12390-2	-	3.69
	111	EN 12390-2	-	4.16

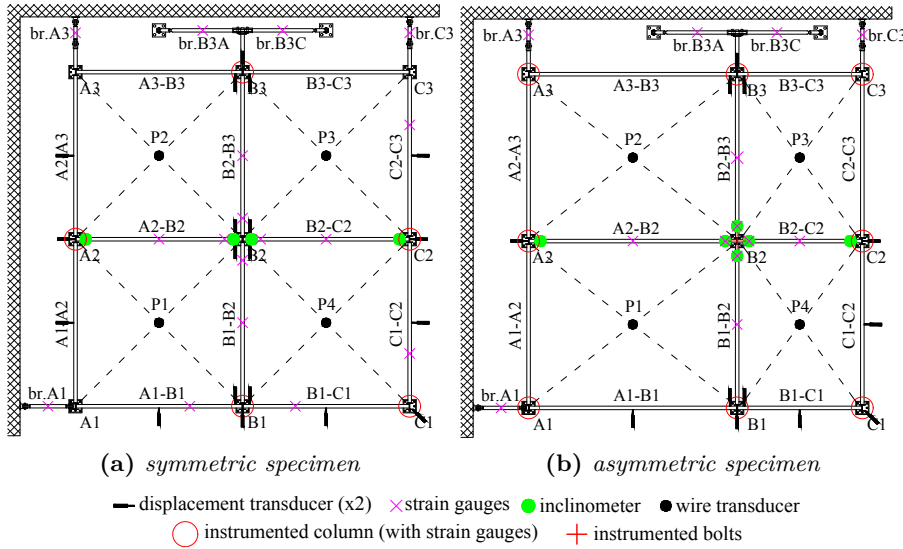


Figure 4.6: The instrumentation set-up

The testing procedure

The tests comprises the following phases, as illustrated in Figure 4.7. At first, the hydraulic actuator was activated and the propping system removed. Afterwards, live loads were applied onto the slab (*Step 1*) by using bags filled with sand to approximate the uniform factored design load of 9.0 kN. This load takes account of finishes, partitions and variable loads, and defines the condition before the column's removal. Each bag was weighted before being placed on the slab surface in two layers. The first layer was uniformly distributed on the complete surface of the slab (153.26 m²), while the second layer acts onto the smaller area as illustrated in Figure 4.8. The specimens under full gravity loading are shown in Figure 4.9. In Table 4.5 are reported the values of the loading applied by each layer. *Step 2* simulated the column removal by reducing the hydraulic pressure of the jack down to zero. The

column removal was simulated quasi-statically by applying a displacement of about 1.4 mm/min. Finally, after a stabilization phase, in *Step 3*, a tensile force was applied at the central node by means of the actuator, and incremented up to 'collapse'. The aim was to quantify the residual strength of the structure after column removal. When two bolts in one end-plate joint of the central node fractured, both in the *symmetric* and *asymmetric* specimen, the collapse condition was assumed to be achieved, and the tests stopped.

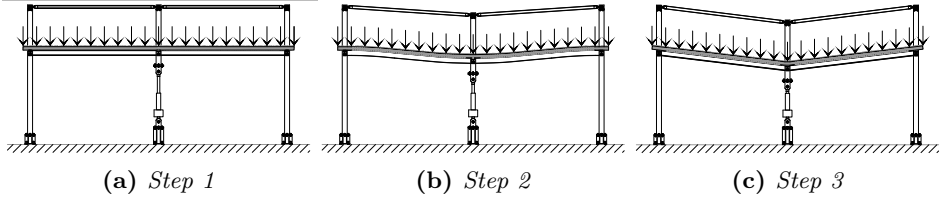


Figure 4.7: The testing phases

Table 4.5: Vertical load applied onto the slab

	Total load (kN)		Area (m ²)		Distributed load (kN/m ²)	
	Symm.	Asymm.	Symm.	Asymm.	Symm.	Asymm.
1 st layer	894.31	932.14	153.26	153.26	5.83	6.08
2 nd layer	440.47	392.22	75.01	74.60	5.87	5.26

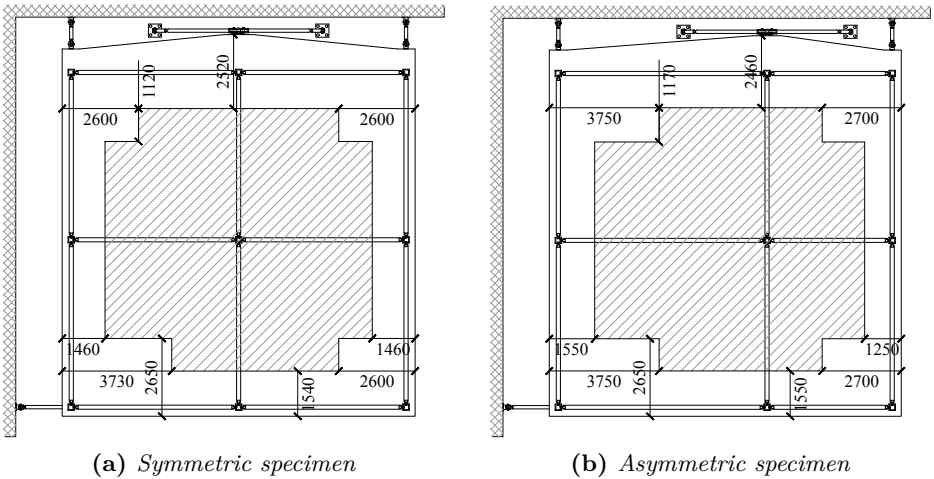


Figure 4.8: The second layer of bags (units in mm)

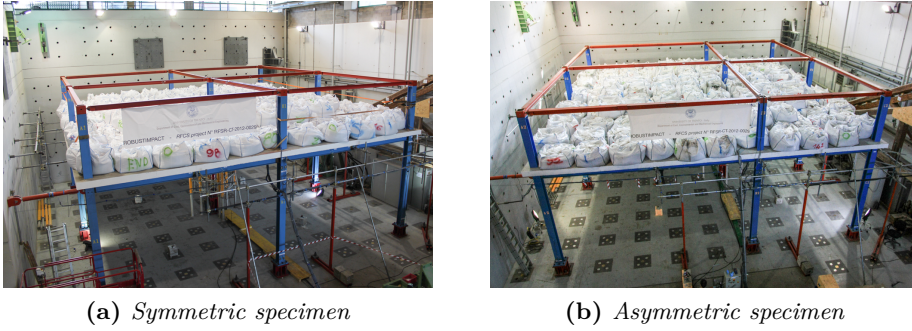


Figure 4.9: The specimens under full gravity loading

Overall response of the specimens

The load-deflection relationship of the 'central column' that characterizes the overall response of the frames is illustrated in Figure 4.10. In the graph, compression forces are positive, while tensile forces are negative. The analysis of the behaviour highlights the testing phases and the corresponding values of the load and deflection at the end of each phase, as reported in Table 4.6.

The response in the three loading phases clearly pointed out the importance of the beam-to-column joints and of the concrete slab. The structural behaviour of the two tested frames was fairly similar, although the *asymmetric* frame exhibited a stiffer response than the *symmetric* one. At the end of the test (*step 3*), the tensile force applied by the actuator to the central node was of 386.30 kN in the *asymmetric* specimen and of 300.85 kN in the *symmetric* one, with the same deflection (about 300 mm). The critical role of the joints' ductility is confirmed; both tests achieved 'collapse' due to the fracture of two bolts in the bottom row in the joint that connects the internal beam B2-B3 to the central column B2 (Figure 4.11). More details on the performance of the various frame components are gathered in the following:

- extensive cracking of the concrete evolved during the loading steps providing a picture of the resistance mechanism of the slab. The slab significantly contributed to the loads redistribution from the damaged to the undamaged parts of the frames, thus allowing a robust structural response. At 'collapse', a visual inspection of the concrete slab revealed on the top side a series of concentric compressive rings, typical of slabs where membrane forces started to be activated (Figure 4.12(a));
- inelastic behaviour developed in the vicinity of the beam-to-column joints, where significant deformations of the end-plates, of the column

web panels, of the steel beams and of the rebars were observed and measured;

- splitting of the slab occurred behind the joints with external columns (Figure 4.12(b)).

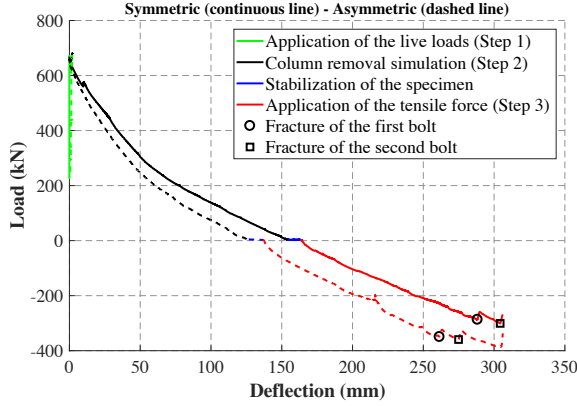


Figure 4.10: Load-deflection relationship of the tested sub-frames

Table 4.6: Load-deflection of the central column at the end of each phase

Testing phase	Load (kN)		Deflection (mm)	
	Symm.	Asymm.	Symm.	Asymm.
End of propping system removal	228.55	250.39	0.00	0.00
End of loading phase	669.25	681.23	0.00	0.00
End of column removal	2.81	4.44	154.60	126.70
End of stabilization phase	2.02	2.90	163.80	136.80
Collapse of the 1 st bolt	-286.80	-349.62	288.35	261.38
Collapse of the 2 nd bolt	-300.85	-359.45	304.30	274.89
End of the test	-300.85	-386.30	304.30	303.64

The joints remarkably contribute to the load redistribution in case of a column loss up to the development of the catenary action in the internal beams. The joints in the central node reached important rotations and axial force demands (Zandonini et al.) as confirmed by the results in the central beams, and have turned out as the most critical elements. The performance of the steel connections are consistent with the studies carried out by Yang and Tan [2013a], who tested common types of bolted-steel beam-to-column connections under a central-column-loss scenario in 2D frames. The research work on T-stub elements provides useful information about reliable end-plate design. Complementary tests on T-stubs extracted from the sub-frames connections were performed as part of the 'RobustImpact' project (Baldassino et al. [2016]).

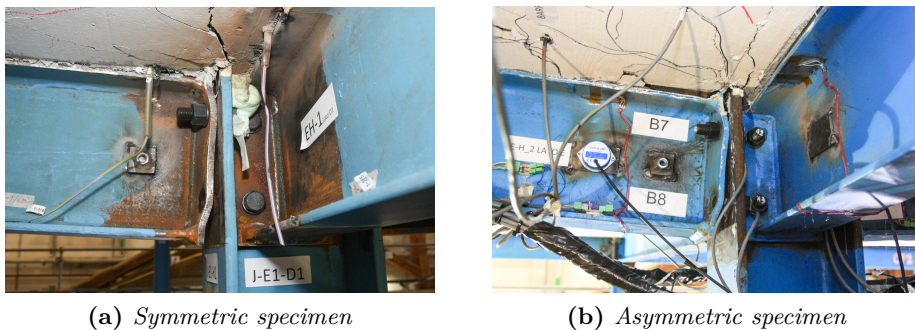


Figure 4.11: The central node at 'collapse'

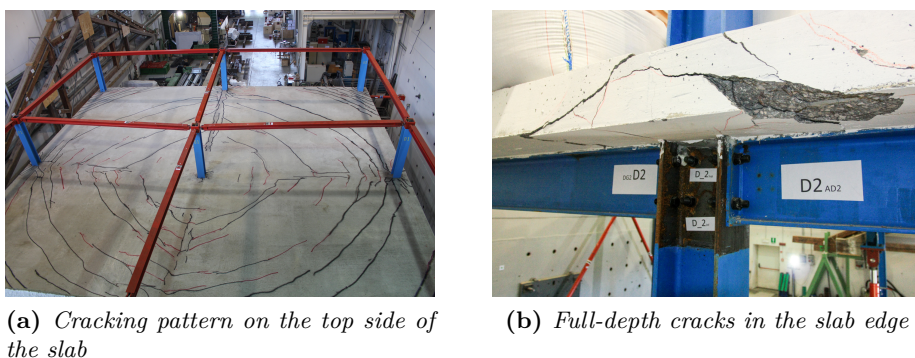


Figure 4.12: The slab at 'collapse'

4.2.3 Concluding remarks about the experimental programme

This section presented the experimental tests conducted at the University of Trento in the framework of an European project on robustness of steel and concrete composite structures. The studies comprise two full-scale tests on 3-dimensional structures subjected to a column loss. The tests aimed at investigating the role of the beam-to-column connections and of the concrete slab in developing alternate load paths, preventing the progressive collapse associated with a column removal.

The specimens, designed according to the Eurocodes, were extracted from reference building structures object of the studies and were characterized by different column layout, symmetric with respect to both the plan directions and symmetric with respect to the transverse direction only. The specimens were loaded at the full factored design load and the collapse of the central column, replaced by an hydraulic actuator, was then simulated. Afterwards, a tensile force was applied at the central column location to assess the residual strength of the systems.

Both the *symmetric* and *asymmetric* specimens achieved 'collapse' due to

the fracture of two bolts in the bottom row of a joint in the central column. The comparison of the load-deflection relationship of the 'removed column' between the two tested frames showed a similar structural behaviour, although a stiffer response of the *asymmetric* specimen is highlighted. The slab significantly contributes to the load redistribution from the damaged to the undamaged parts of the frames, thus allowing a robust structural response. At the end of the tests, a visual inspection of the slab revealed concentric compressive rings on the top side of the slab, typical of slabs where membrane forces starts to be activated. The outcomes of the tests confirm that the Eurocodes specifications seem a good base for the design of a robust composite frame.

4.3 Modelling and calibration of the numerical tool

The finite element models were developed by means of the software Abaqus/CAE (Simulia Dassault Systems). Although the dynamic response of structures subjected to column loss, quasi-static analyses were conducted in order to compare the results with the experimental tests. Explicit analyses have been adopted due to several mechanical and geometrical nonlinearities and to overcome convergence difficulties in static solver. However, the explicit dynamic solver requires high computational resources and often produces unstable structural response (Yang and Tan [2011]), the dynamic effects were neglected by suitably selecting the '*time period*' in the step definition and assigning a gradual application of the loads.

4.3.1 Description of the finite element models on the sub-frames

The sub-frames were modelled with two types of steel beam-to-column connections: pin connections and semi-rigid connections. This choice was made to find a solution that allowed a good approximation of the results compared to the experimental ones and a satisfactory modelling and computational burden. The aim of the study is to model the full-frame structures as the sub-frames validated with the tests.

The steel elements (beams, columns, lateral restraints) were modelled as 1D beam element (*BEAM 'B31'*: 2-node linear beam in space), while the concrete slab as 2D shell element of approximately 250×250 mm side (*SHELL 'S4R'*: 4-node, stress/displacement shell element with reduced integration). The finite element models of the *symmetric* and *asymmetric* specimens were built up with reference to the structural elements and dimensions described in 4.2.2 without considering the geometrical imperfections. Cross sectional dimensions have been assigned to the structural elements as reported in Table 4.7. The geometrical properties of the sub-frames are illustrated in

Figure 4.4, while the assembled models are shown in Figure 4.13.

The concrete slab was adequately partitioned as in Figure 4.14 to consider the position of the reinforcement (Baldassino et al. [2018]). This allows a satisfactory approximation of the rebars layout. It is important to notice that the shell elements selected account for the rebars as embedded within. Table 4.8 and Table 4.9 report the cross sectional area and spacing of the reinforcements for each section of the slab, both for the *symmetric* and *asymmetric* frame, respectively. The steel beams and the reinforced concrete slab are rigidly coupled together using 'Tie' multi-point constraints to simulate the complete shear interaction.

Table 4.7: Cross-section of the structural elements

Structural element	Cross-section
Columns	HEB220
Beams	IPE240
Crowning beams	HEA140
Lateral restraints	Pipe 114.3 × 6.3 mm
Slab	165.0 mm

Table 4.8: Rebars amounts in the slab sections - symmetric sub-frame (units in mm)

Slab section	X direction Bottom	Y direction Bottom	X direction Top	Y direction Top
A	$\phi 10/150$	$\phi 10/150$	$\phi 10/150$	$\phi 10/150$
B	2 $\phi 10/150$	2 $\phi 10/150$	$\phi 10/150$	$\phi 10/150$
C	$\phi 10/150$	$\phi 10/150$	$\phi 10 + \phi 16/150$	$\phi 10/150$
D	$\phi 10/150$	$\phi 10/150$	$\phi 10/150$	$\phi 10 + \phi 16/150$
E	$\phi 10/150$	$\phi 10/150$	2 $\phi 10/150$	$\phi 10/150$
F	$\phi 10/150$	$\phi 10/150$	$\phi 10/150$	2 $\phi 10/150$
G	$\phi 10/150$	$\phi 10/150$	$\phi 10 + \phi 16/150$	$\phi 10 + \phi 16/150$
H	2 $\phi 10/150$	2 $\phi 10/150$	$\phi 10 + \phi 16/150$	$\phi 10 + \phi 16/150$
I	2 $\phi 10/150$	$\phi 10/150$	$\phi 10 + \phi 16/150$	$\phi 10 + \phi 16/150$
L	$\phi 10/150$	2 $\phi 10/150$	$\phi 10 + \phi 16/150$	$\phi 10 + \phi 16/150$

The steel-beam-to-column connections, illustrated in Figure 4.3, were modelled with different levels of accuracy. In all models, the slab at the node was either continuous (inner node) or detailed to ensure slab-column interaction. Firstly, hinges at the intersection of the columns with the steel beams were assumed. The results, reported in the following section, showed that the numerical models approximate sufficiently well the experimental load-deflection relationship of the 'central column' and the load redistribution among the columns, but the approximation of the joints response was far from being satisfactory. Then, in accordance to the Eurocode (EN1993-1-8 [2005]), the component method was applied. Therefore, non-linear translational springs simulating the response of each mechanical component were

Table 4.9: Rebars amounts in the slab sections - asymmetric sub-frame (units in mm)

Slab section	X direction Bottom	Y direction Bottom	X direction Top	Y direction Top
A	$\phi 10/150$	$\phi 10/150$	$\phi 10/150$	$\phi 10/150$
B	2 $\phi 10/150$	2 $\phi 10/150$	$\phi 10/150$	$\phi 10/150$
C	$\phi 10/150$	2 $\phi 10/150$	$\phi 10/150$	$\phi 10/150$
D	$\phi 10/150$	$\phi 10/150$	$\phi 10 + \phi 16/150$	$\phi 10/150$
E	2 $\phi 10/150$	2 $\phi 10/150$	$\phi 10/150$	$\phi 10 + \phi 16/150$
F	$\phi 10/150$	$\phi 10/150$	$\phi 10/150$	2 $\phi 10/150$
G	$\phi 10/150$	2 $\phi 10/150$	2 $\phi 10/150$	$\phi 10/150$
H	$\phi 10/150$	$\phi 10/150$	2 $\phi 10/150$	$\phi 10/150$
I	$\phi 10/150$	$\phi 10/150$	2 $\phi 10/150$	$\phi 10 + \phi 16/150$
L	$\phi 10/150$	$\phi 10/150$	$\phi 10 + \phi 16/150$	$\phi 10 + \phi 16/150$
M	2 $\phi 10/150$	$\phi 10/150$	$\phi 10 + \phi 16/150$	$\phi 10 + \phi 16/150$
N	$\phi 10/150$	2 $\phi 10/150$	$\phi 10 + \phi 16/150$	$\phi 10 + \phi 16/150$
O	2 $\phi 10/150$	2 $\phi 10/150$	$\phi 10 + \phi 16/150$	$\phi 10 + \phi 16/150$
P	$\phi 10/150$	2 $\phi 10/150$	2 $\phi 10/150$	$\phi 10 + \phi 16/150$

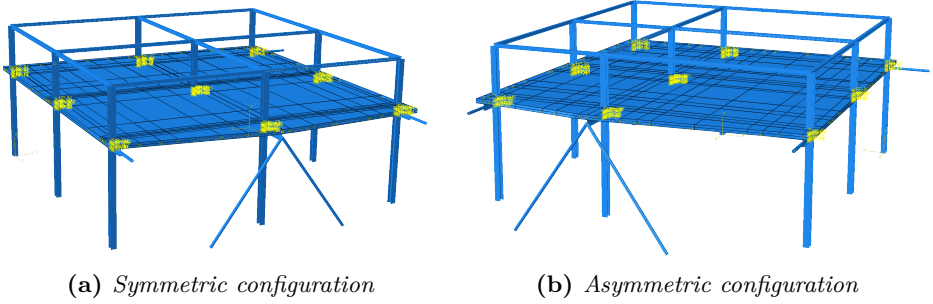


Figure 4.13: The finite element models on the sub-frames

implemented as illustrated in Figure 4.15. The properties of the springs in tension and in compression (stiffness and strength) were calculated according to the component method proposed by the EN1993-1-8 [2005], taking also into account the outcomes of Steenhuis et al. [1998] and Baldassino et al. [2016]. In order to simulate the response during all the loading phases, the model need to comprise spring elements acting in tension only or in compression only. These springs have a different location, and are inactive when the stress state inverts. Finally, an additional spring was incorporated in correspondence to the top layer of reinforcement in the concrete slab at the back of external column (k_{rebars} in Figure 4.15(b)) in order to consider the column-slab force interaction (Figure 4.17). A strut-and-tie model (Figure 4.18) allowed calculating the stiffness of the 'rebars' spring as described in Pardatscher [1996]. Comparing the numerical and experimental results, the model with non-linear springs appeared to be more than satisfactory for

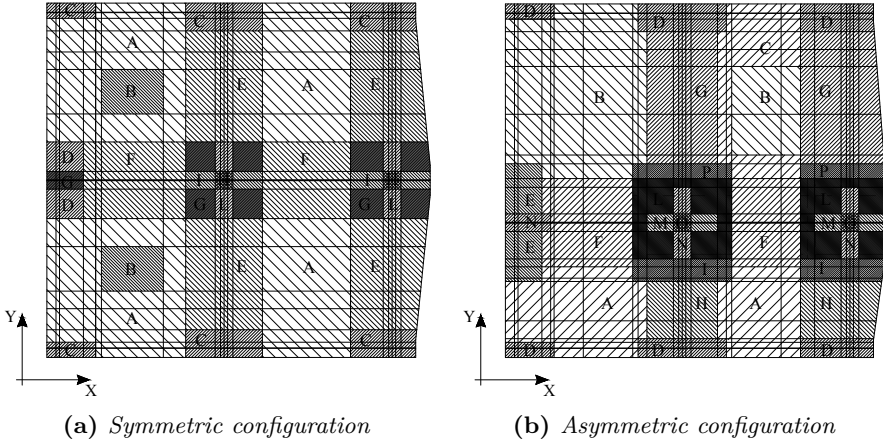


Figure 4.14: Partition of the concrete slab

getting the key parameters describing the frames response in all the loading phases. Furthermore, the model assumes: i) pin connections for the crowning beams as well as for the lateral restraints to the steel frame; ii) rigid connections at the columns base.

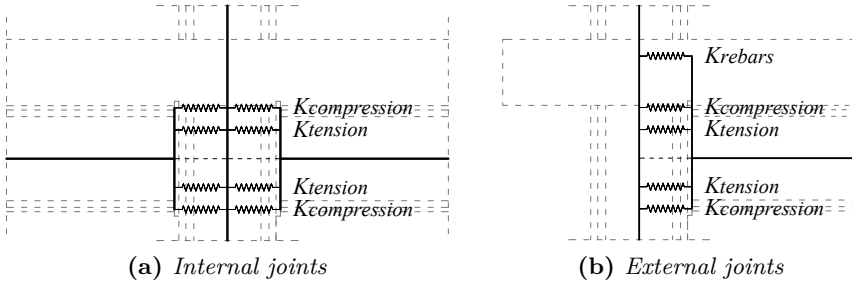


Figure 4.15: Model of the beam-to-column connections

As to the materials, a simplified elastic-plastic with hardening law is assumed for steel. The average yield stress and the average ultimate tensile strength were obtained from experimental tests on material samples of the columns, beams and rebars as described in 4.2.2. Nominal values of yield stress is assumed for the steel of the crowning beams and of the lateral restraints. Furthermore, the elastic modulus is assumed as reported in Table 4.10.

Concrete is modelled implementing the 'Concrete Damaged Plasticity' model that allows simulating the inelastic behaviour of the concrete both in compression and in tension. Shrinkage and creep phenomena are not introduced in the computation. The compression and tension strength and the elastic modulus of the concrete, reported in Table 4.11, were obtained from ex-

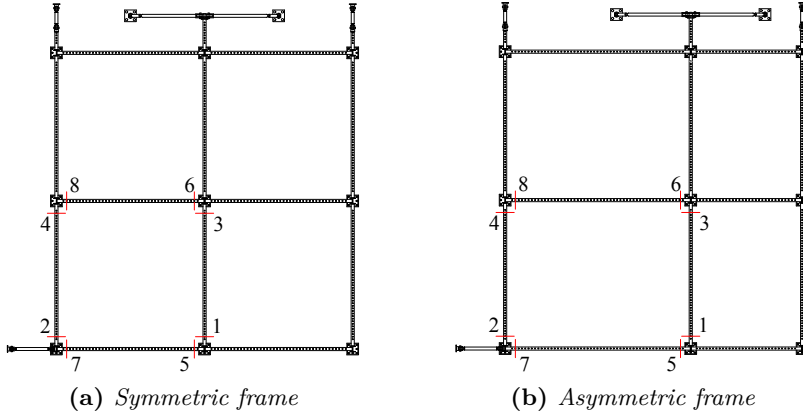
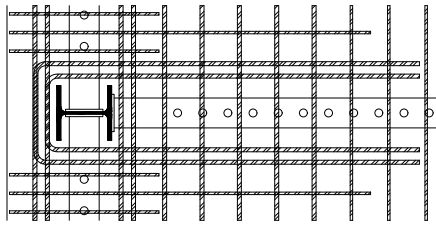


Figure 4.16: Position and nomenclature of the joints



(a) Reinforcement details near the external column



(b) Cracks in the concrete

Figure 4.17: External columns-slab interaction

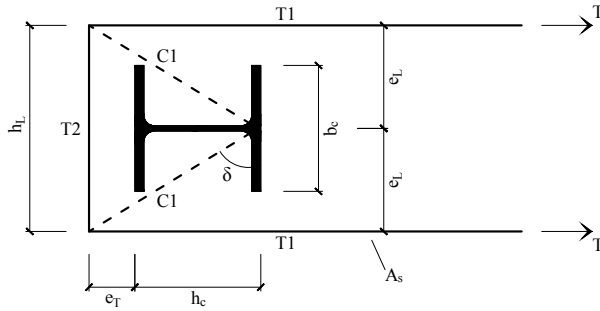


Figure 4.18: The strut and tie model

perimental tests on material samples (Zandonini et al.). The stress-strain relationship of the concrete in compression follows the recommendations of the EN1992-1-1 [2004], while in tension a simplified relation proposed by Allam et al. [2013] allows considering the tension stiffening effect. Since the elastic modulus of the concrete measured by the tests was far from the

value proposed by the EN1992-1-1 [2004], the parameters ϵ_{c1} and ϵ_{cu1} were carefully modified in order to compute the stress-strain relationship for the concrete in compression.

Table 4.10: *Steel properties*

Structural element	f_y (MPa)	f_u (MPa)	E_s (MPa)
Columns	303.3	440.3	210000
Beams	409.3	540.7	210000
Crowning beams	235.0	1.1 f_y	210000
Lateral restraints	355.0	1.1 f_y	210000
Rebars	496.0	586.0	200000

Table 4.11: *Concrete properties*

	<i>Symmetric frame</i>	<i>Asymmetric frame</i>
f_{cm} (MPa)	54.56	47.59
f_{ctm} (MPa)	4.25	4.16
E_{cm} (MPa)	19000	21070
ϵ_{c1} (‰)	3.2	3.1
ϵ_{cu1} (‰)	3.7	3.9

In the finite element analysis, the material model has to be defined by true stress and true strain relationship (Yang and Tan [2011]). Therefore, the engineering stress and strain values have to be converted in true stress and strain by means of the following equations:

$$\sigma_{true} = \sigma_{eng} (1 + \epsilon_{eng}) \quad (4.1)$$

$$\epsilon_{true} = \ln (1 + \epsilon_{eng}) \quad (4.2)$$

A damage model in the concrete was implemented. For this purpose, it is necessary to define the compression and tension damage parameters in the 'Concrete Damaged Plasticity' model. The damage variables are assumed to be dependent on the stress and are equal to zero before the peak stress (Yu et al. [2010b]). In particular, for concrete under uniaxial compression, the damage parameter after the peak stress is given by:

$$d_c = 1 - \frac{\sigma_c}{f_{cm}} \quad (4.3)$$

in which σ_c is the stress of the concrete in the descending branch. Similarly to the compression case, the tension damage parameter is calculated as:

$$d_t = 1 - \frac{\sigma_t}{f_{ctm}} \quad (4.4)$$

In Figure 4.19 the stress-strain relationship of the concrete in compression and in tension for both the symmetric and asymmetric frame are illustrated.

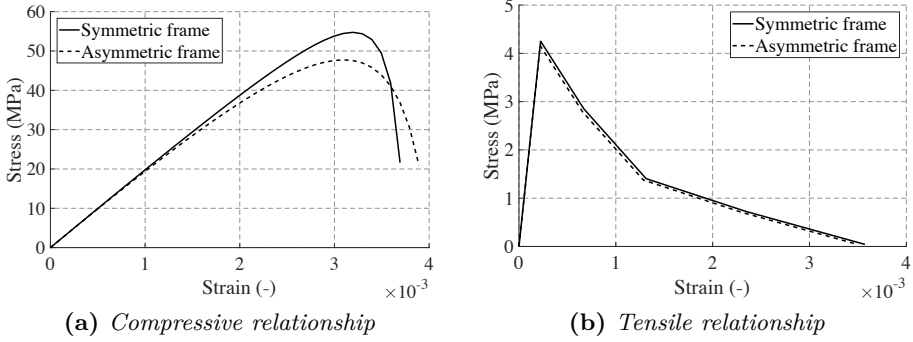


Figure 4.19: Stress-strain relationship of concrete

The finite element model is composed of three sub-models in order to compare the results with the experimental ones and to define a loading step as in the tests. In particular, the three sub-models are:

- loading G: this model allowed defining the stress and deformation state of the structures relative to the dead loads that were not measured during the experimental test because of the instruments were installed at the end of the constructional phases;
- loading G+Q: this model allowed determining the reaction force in the central column due to dead and live loads;
- test G+Q: finally, this model simulated the column removal as in the tests.

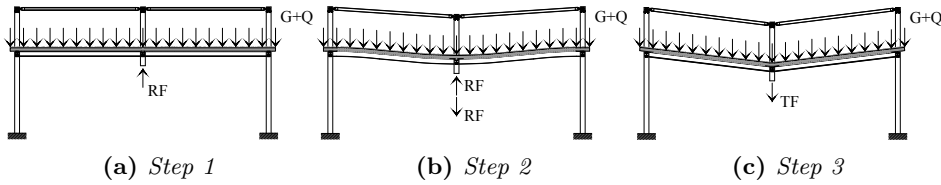
The sub-model 'test G+Q' is composed of following loading steps (4.2.2):

- *Step 1*: application of the dead and live loads on the frames and simultaneously the reaction force (RF) in the central column (determined in the *loading G+Q* model), as illustrated in Figure 4.20 (a). The loads applied on the structures are reported in Table 4.12 and are in accordance with the experimental quantities described in 4.2.2;
- *Step 2*: simulation of the column removal by applying an opposite reaction force in the central column (Figure 4.20 (b));

Table 4.12: Loads applied on the sub-frame structures

	Dead loads	Live loads	
		<i>Symm. frame</i>	<i>Asymm. frame</i>
Steel density (kN/m^3)	78.50	-	-
Concrete density (kN/m^3)	24.75	-	-
Layer 1 (kN/m^2)	-	5.83	6.08
Layer 2 (kN/m^2)	-	5.87	5.26

- *Step 3*: application of the tensile force (TF) in the central column to evaluate the residual strength of the frames as in the tests (Figure 4.20 (c)).

**Figure 4.20:** The loading steps

4.3.2 Comparison of the results with the experimental tests

In general, the numerical simulations are in a rather good agreement with the test results, although the models with pin connections present less accuracy in terms of some parameters than the semi-rigid connections models. Figure 4.21 relates to the overall response of the *symmetric* and *asymmetric* frame: the deflection at the central node is plotted with respect to the load supported by the central column. It is apparent that the numerical models approximate with good accuracy the experimental relationships, both for the *symmetric* and *asymmetric* frame; in the first phase of the column removal the numerical analysis with semi-rigid connections provide a stiffer response. The numerical-pin models overestimate the load at the end of the loading phase (*step 1*), and underestimate the deflection at the end of *step 3*. Despite these discrepancies, the FEM models with pin connections seem to be satisfactory for the overall structural response, even if the models with semi-rigid connections are more accurate. Figure 4.22 and Figure 4.23 show the deformed shape of the *symmetric* and *asymmetric* sub-frame, respectively, at the end of the three loading steps obtained from the numerical models with semi-rigid beam-to-column connections.

Other important parameters in order to validate the numerical models are: axial forces in the columns, beam-to-column joints rotation and stress dis-

tribution in the slab. The parameters are plotted with respect to the load supported by the central column.

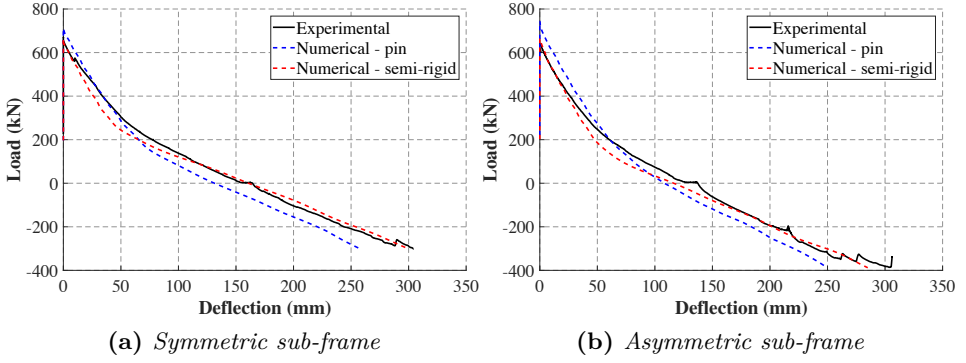


Figure 4.21: Comparison of the load-deflection relationship of the central node between experimental and numerical analysis

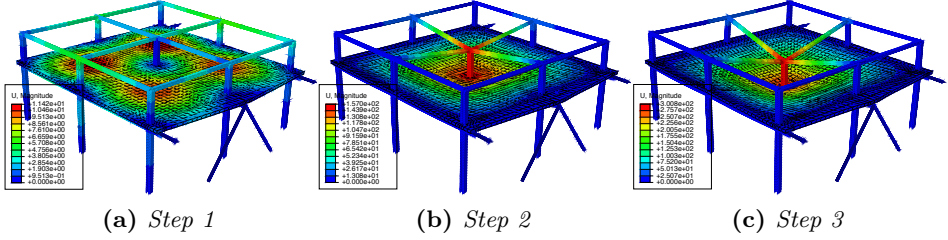


Figure 4.22: Vertical displacement of the symmetric sub-frame - semi-rigid numerical model

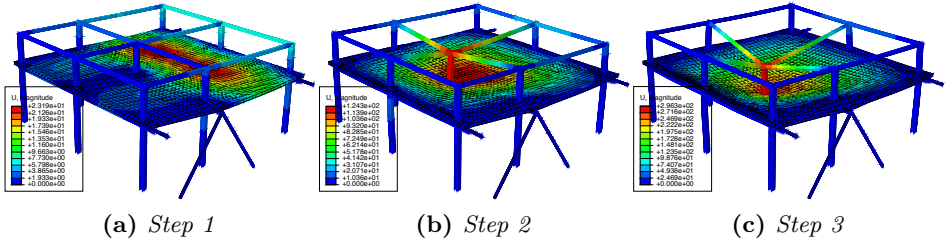


Figure 4.23: Vertical displacement of the asymmetric sub-frame - semi-rigid numerical model

Thrusts in the columns

The distribution of the axial forces in the columns (named as in Figure 4.24) at the end of the three loading steps is illustrated in Figure 4.25.

This parameter allows understanding the load redistribution from the damaged to the undamaged part of the system after the column removal.

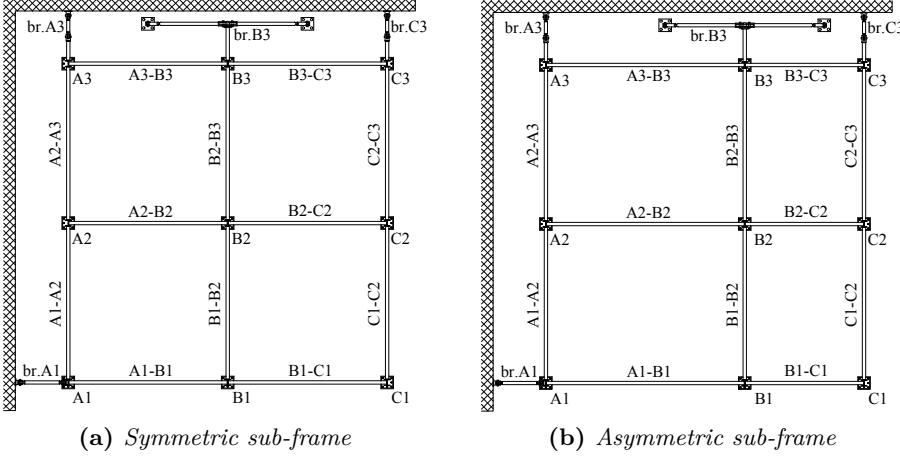


Figure 4.24: Nomenclature of the beams and columns in the sub-frames

At the end of *step 1* (Figure 4.25 (a) and (b)), the load supported by the central column and measured by the load cell was of 669.9 kN and 681.2 kN for the *symmetric* and *asymmetric* frame, respectively. Afterwards, at the end of *step 2*, the central column was completely lost. Columns B1, A2 and C2 (Figure 4.25 (c)) carried about twice the axial force acting before the column removal. The force in column B3 was the greatest due to the nearby restraining truss system br.B3, while corner column C1 did not show particular variations. The asymmetry of the frame influenced significantly the axial force at the base of corner columns C1, A3, and C3 and of lateral column C2 (Figure 4.25 (d)), as following: i) corner column C1, compressed at the end of the loading phase, became stretched at the end of the column removal; ii) corner columns A3 and C3 carried about half the force acting before *step 2*; iii) the force in column C2 at the end of *step 2* was about three times the force acting before the column loss. Lateral columns B1 and B3 did not showed particular differences from the same columns in the *symmetric* frame, while the column A2 carried less than two times the force acting at the end of the loading phase (*step 1*). At the end of *step 3*, the tensile force applied by the actuator to the central node (300.9 kN and 386.3 kN for the *symmetric* and *asymmetric* frame, respectively) has redistributed in the other columns as shown in Figure 4.25 (e) and (f). Focusing on the *symmetric* sub-frame, the force in columns B1 and C2 increased up to about 370 kN and in columns A2 and B3 to 330.8 kN and 635.5 kN, respectively. Corner column C1 substantially did not change the force acting at the end of *step 1*. In the *asymmetric* structure, the force in columns B1 and C2 increased up to about 420 kN and in column A2

to 324.7 kN. Lateral column B3 did not show significant differences from column B3 in the *symmetric* frame, and corner columns A1, A3, and C3 did not show any particular changes in the axial force acting before the application of the tensile force. Column C1 remained stretched also during *step 3*.

The analyses of the axial force in the columns allows drawing important considerations:

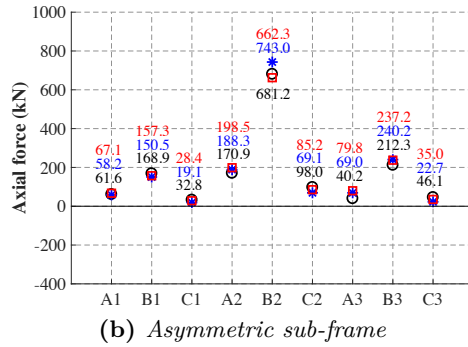
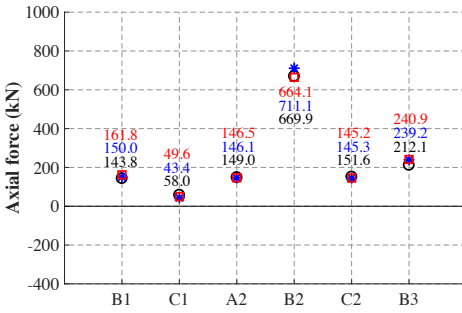
- the concrete slab has a significant role for the load redistribution among the structure. In particular, the columns most influenced by the central column loss are positioned in the frames converging to central column B2 (columns B1, A2, B3, and C2). The corner columns had a very limited axial load variation, and in column C2 (Figure 4.25 (d)) the force passed from compressive to tensile at the end of *step 2*;
- column B3 was the most stressed due to the nearby restraining system, simulating the continuity with the adjacent part of the frame. In the actual full-frame structure, buckling phenomena can occur in this column.

The numerical models are approximating well the thrusts in the columns in all the loading phases. Although there are some discrepancies in the corner columns and in lateral column B3, the numerical models, both with pin and semi-rigid connections, approximate with good accuracy the load redistribution within the structure. Such differences are not significant considering the complexity of the frame behaviour, in particular of the slab.

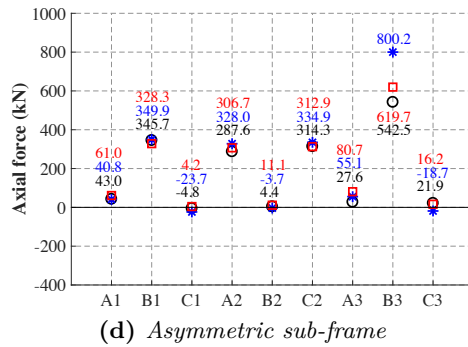
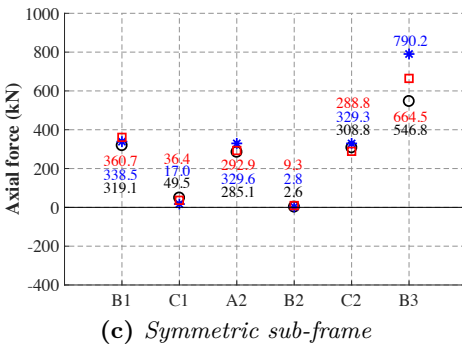
Beam-to-column joints

Both external and central beam-to-column joints, illustrated in Figure 4.26, sustained important rotations at the end of *step 3*. Figures from 4.26 to 4.29 report the comparison of the external joints rotation between experimental and numerical analyses, while Figures from 4.30 to 4.33 refer to central joints. Positive rotation assumptions are illustrated in Figure 4.35. The deformation of the web panel in shear (Figure 4.36 (a)), and the plastic instability of the bottom flange (Figure 4.36 (b)) were the main sources of rotation in the joints B1-B2_B1 and B2-B3_B3. The *symmetric* and *asymmetric* frame exhibited similar results considering the rotation in the external joints. As to the internal joints, the sign reversal of the rotation associated with the column removal is apparent, leading to the change from hogging to sagging bending moment in all connections at the central node, well before the full unloading. Different response is found in joint B2-C2_B2 in the *asymmetric* frame (Figure 4.34 (b)), where the sign reversal of the rotation during *step 2* is not shown due to the asymmetry of the frame. The curves related to joints A2-B2_B2 and B2-C2_B2 in the *symmetric* frame

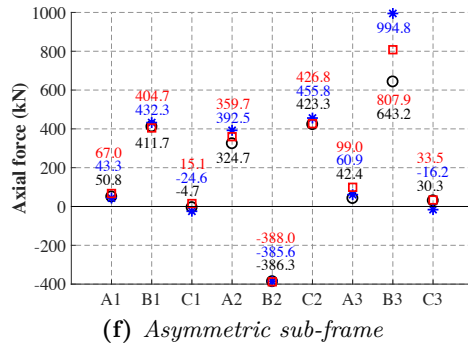
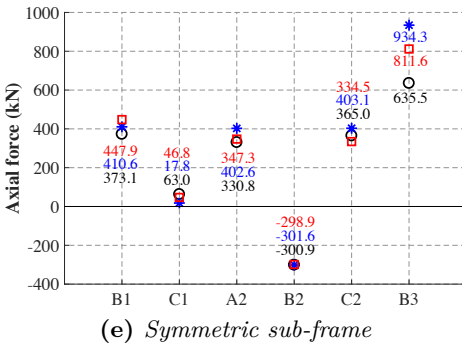
Step 1



Step 2



Step 3



○ Experimental * Numerical - pin □ Numerical - semi-rigid

Figure 4.25: Comparison of the axial force in the columns between experimental and numerical analysis

(Figure 4.32 (a) and Figure 4.34 (a), respectively) confirm the symmetry of the transverse behaviour. The influence of the frame continuity explains the difference in joints B1-B2_B2 (Figure 4.31) and B2-B3_B2 (Figure 4.33). The plastic deformation of the end-plates in bending was the main source of significant rotation capacity (Figure 4.11). It is interesting to note that

joints designed as ductile under monotonic loading were able to achieve important plastic rotations also under the combination of sagging moments and tension axial forces associated with column loss.

As expected, the external joints showed a continuous increase of rotation associated to the central column loss both in the *symmetric* and *asymmetric* frame. The rotation of joint A2-B2_A2 in the *symmetric* frame (Figure 4.28 (a)) is not reported in *step 3* of the test due to an instrument malfunction. As a first appraisal of the comparison between experimental and numerical results, the numerical model with semi-rigid connections approximates more than adequately the joints rotation during all the three steps of the test. In particular, it catches well the sign reversal of the rotation of the central joints and the rotation at the end of *step 3*, important to understand the ductility demand of the connections. The numerical model with pin connections approximates with good accuracy the rotation in the external joints, but the results in the internal connections are not particularly satisfactory; this is the reason why the model with semi-rigid connection is considered more accurate to evaluate the response of the frame. On the other hand, both numerical models with pin and semi-rigid connections do not catch the experimental results in joint B2-B3_B3 (Figure 4.29), where the rotation was influenced by the buckling phenomenon in the beam, which can not be predicted by a FEM model with 1D beam elements.

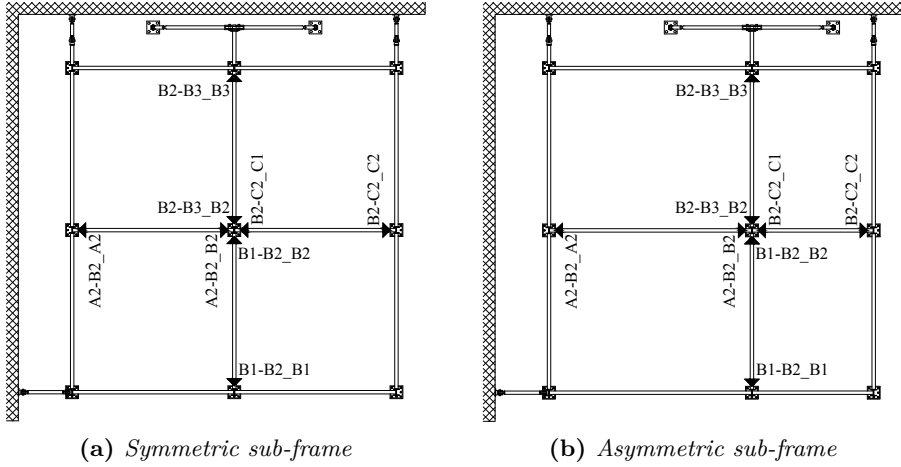


Figure 4.26: Nomenclature of the joints in the sub-frames

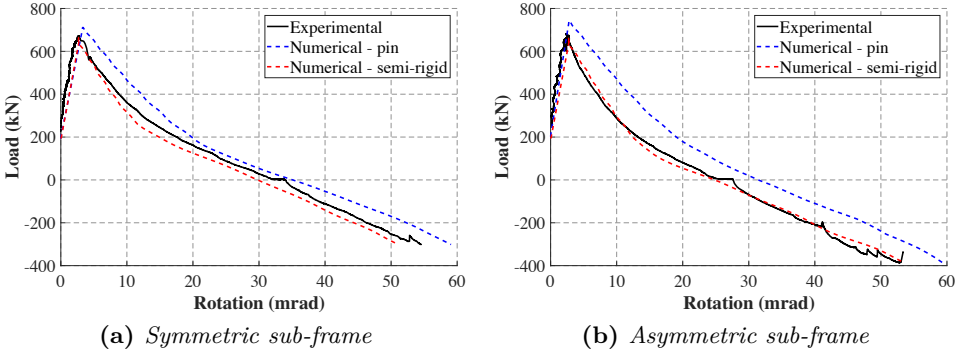


Figure 4.27: Comparison of the rotations in the external joint B1-B2_B1

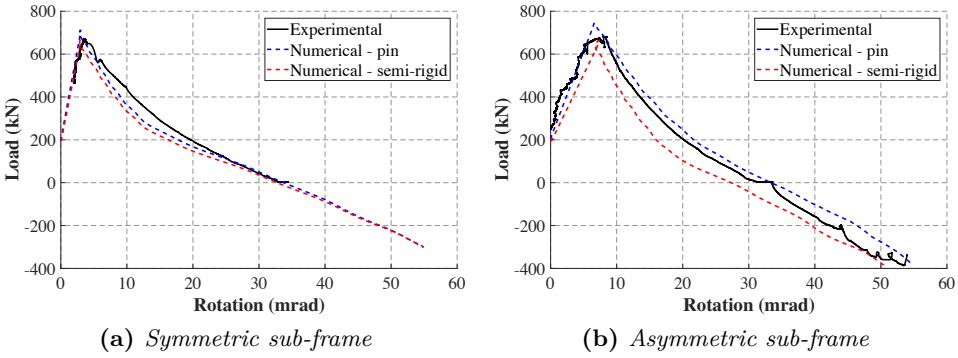


Figure 4.28: Comparison of the rotations in the external joint A2-B2_A2

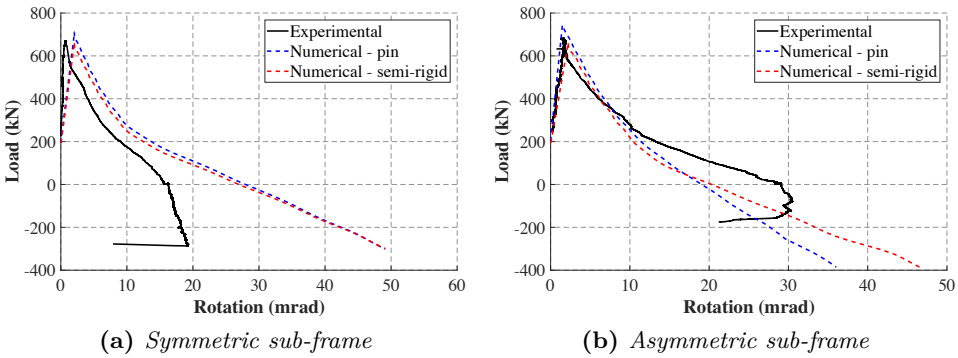


Figure 4.29: Comparison of the rotations in the external joint B2-B3_B3

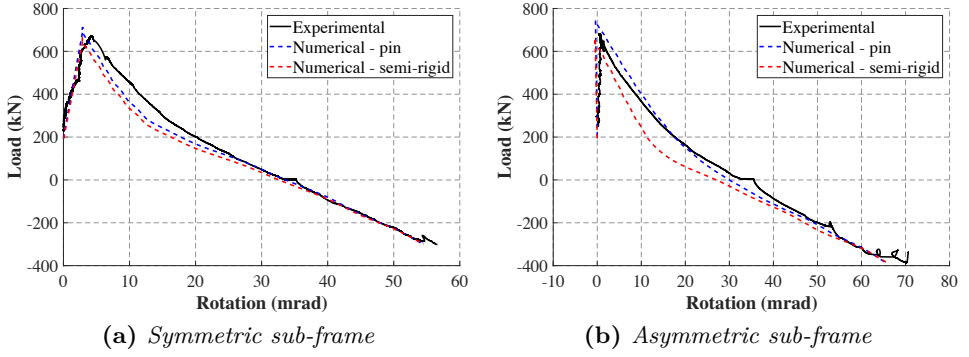


Figure 4.30: Comparison of the rotations in the external joint B2-C2_C2

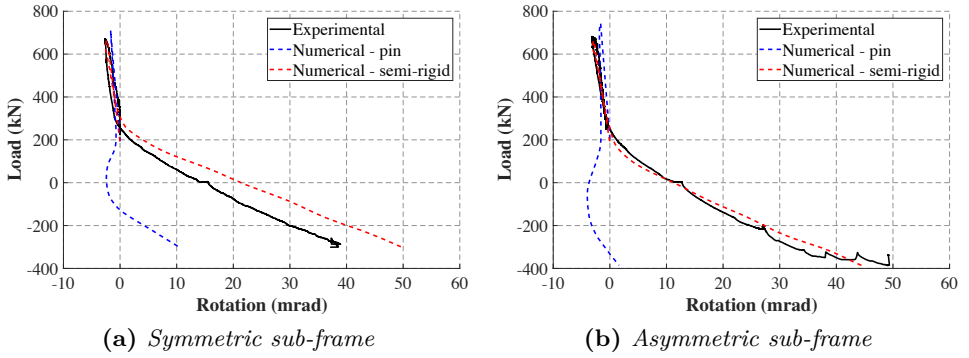


Figure 4.31: Comparison of the rotations in the internal joint B1-B2_B2

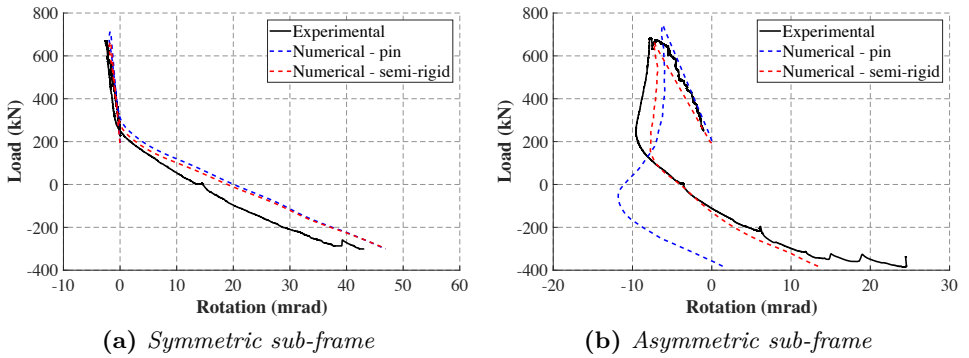


Figure 4.32: Comparison of the rotations in the internal joint A2-B2_B2

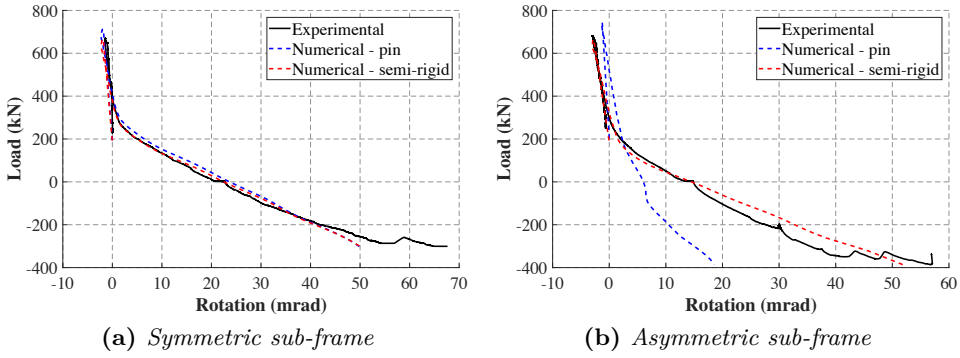


Figure 4.33: Comparison of the rotations in the internal joint B2-B3_B2

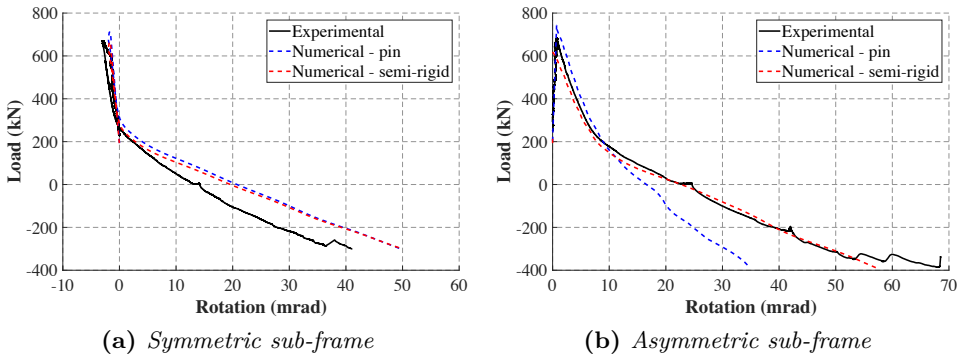


Figure 4.34: Comparison of the rotations in the internal joint B2-C2_B2

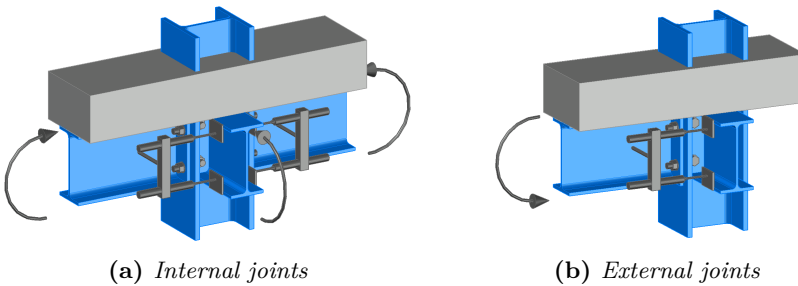
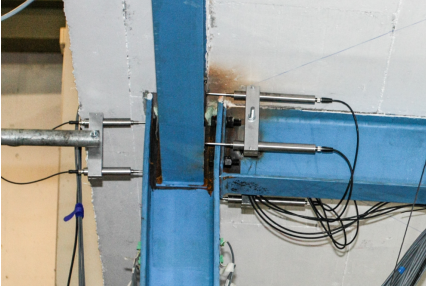
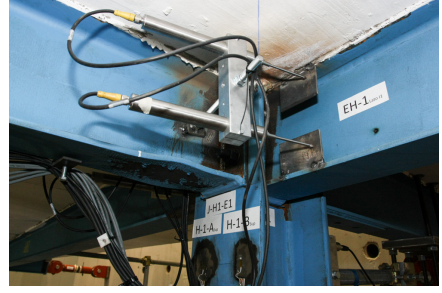


Figure 4.35: Positive sign of joints rotation



(a) Joint B1-B2_B1 - shear deformation of the column web panel



(b) Joint B2-B3_B3 - buckling of the beam flange

Figure 4.36: External joints at the end of the test

The slab

The finite element models can not predict the local behaviour of the slab; however, the cracking patterns and the stress distribution are an index of its overall response. The observed cracking pattern on the bottom side of the slab, illustrated in Figure 4.37 and Figure 4.38 for the *symmetric* and *asymmetric* frame, respectively, revealed fully cracked regions along the diagonals of the four slab panels. The numerical tension damage patterns, shown in Figure 4.39 and in Figure 4.40, obtained from the models with semi-rigid connections, are in accordance with the experimental evidences. Furthermore, the figures show the spread of the damage from the end of *step 2* to the end of *step 3*. The tensile stresses in the bottom side of the slab, illustrated in Figure 4.41, are related to the tension damage pattern. At the end of *step 1*, the asymmetry of the frame influences significantly the stresses in the bottom side (Figure 4.41 (b)); at the contrary, Figure 4.41 (a) highlights the symmetric response of the four slab panels. At the end of *step 2* and *step 3*, the tensile stresses in the regions along the diagonals are null because of the cracks or because of the compressed zones (as illustrated in Figure 4.42).

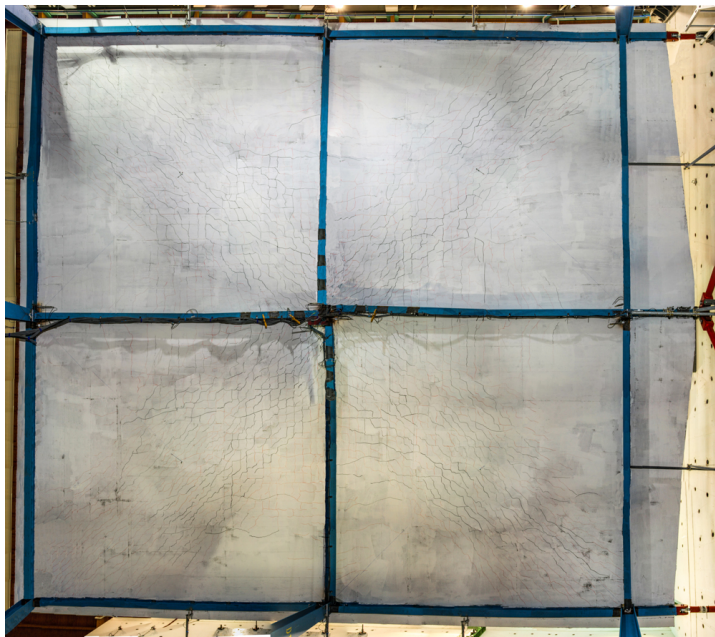


Figure 4.37: *The cracking pattern on the bottom side of the slab - symmetric sub-frame*



Figure 4.38: *The cracking pattern on the bottom side of the slab - asymmetric sub-frame*

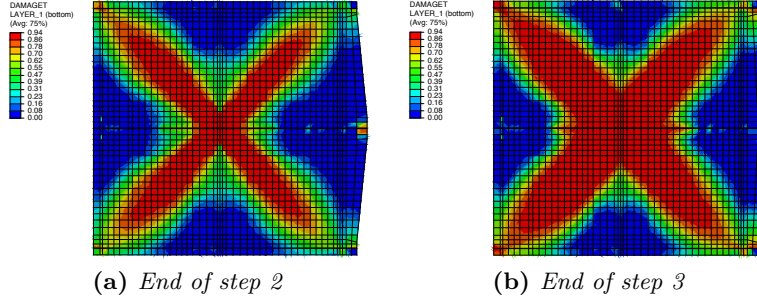


Figure 4.39: Tension damage pattern on the bottom side of the slab - symmetric sub-frame

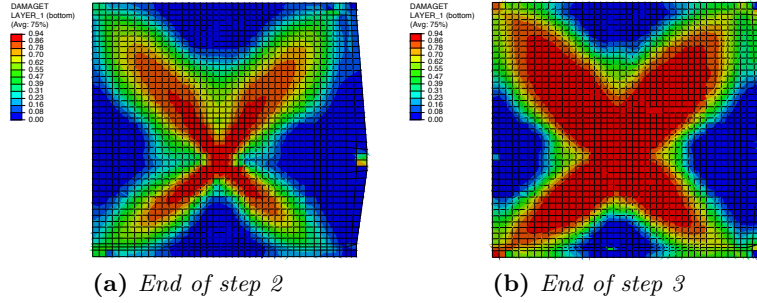
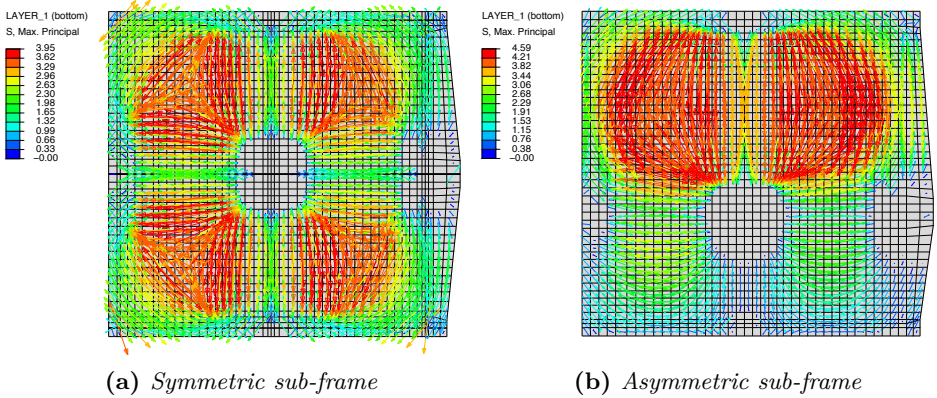


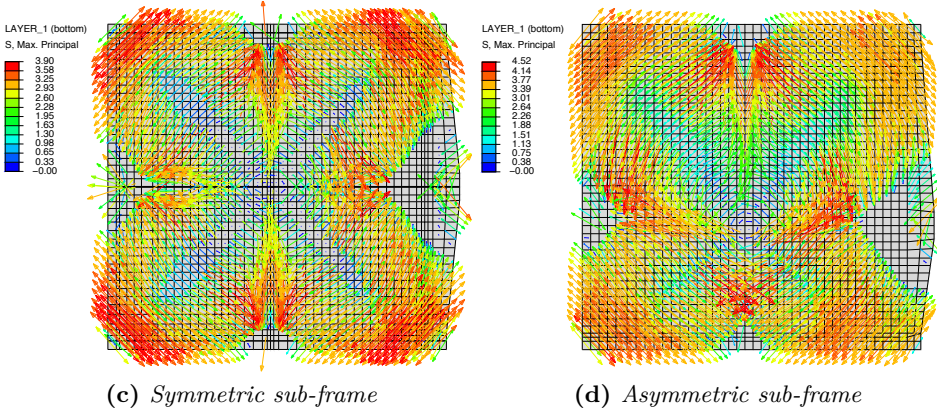
Figure 4.40: Tension damage pattern on the bottom side of the slab - asymmetric sub-frame

The observed cracking pattern on the top side of the concrete slab at the end of the test is illustrated in Figure 4.43. A sort of concentric compressive rings around the central column are apparent, both in the *symmetric* and *asymmetric* frame. The damage patterns on the top side, shown in Figure 4.44 and Figure 4.45, highlight cracked regions around the supports, mainly located in correspondence of column B3, simulating the continuity with the adjacent part of the structure. The tensile stresses during the loading phase (*step 1*) are in correspondence of the supports and of the central beams (Figure 4.46 (a) and (b)), while at the end of the column removal (Figure 4.46 (c) and (d)) the stretched regions are close to the lateral beams. The pattern of compressive stress obtained from the numerical models (Figure 4.47), provides a similar picture of the experimental response. The comparison between the experimental and numerical outcomes in the concrete slab allows to conclude that the FEM models enable predicting the overall response of the slab, both at the bottom and top side. Furthermore, the differences of the slab response between the model with pin and semi-rigid connections are not relevant, and, for this reason, they are not reported in this work.

Step 1



Step 2



Step 3

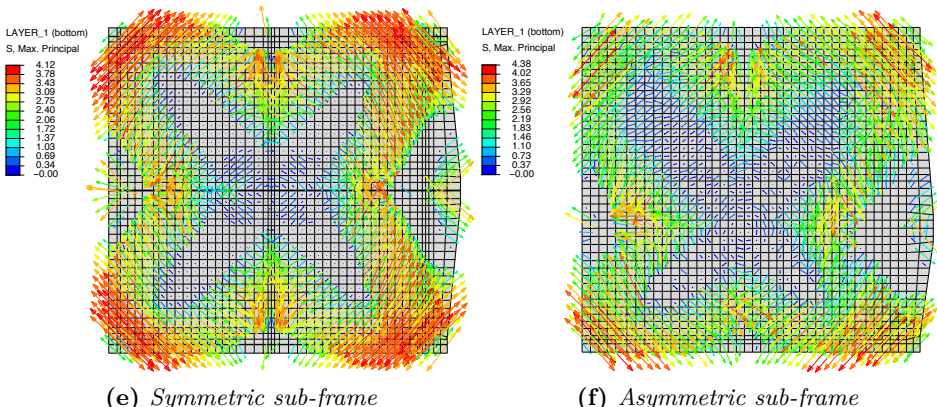
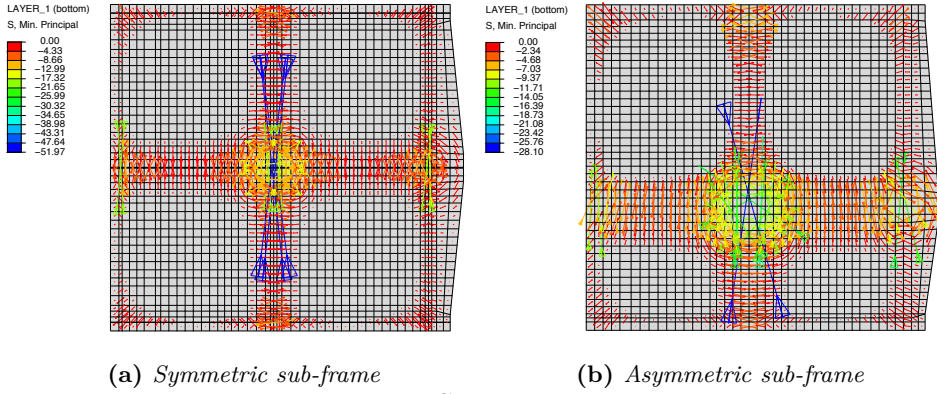
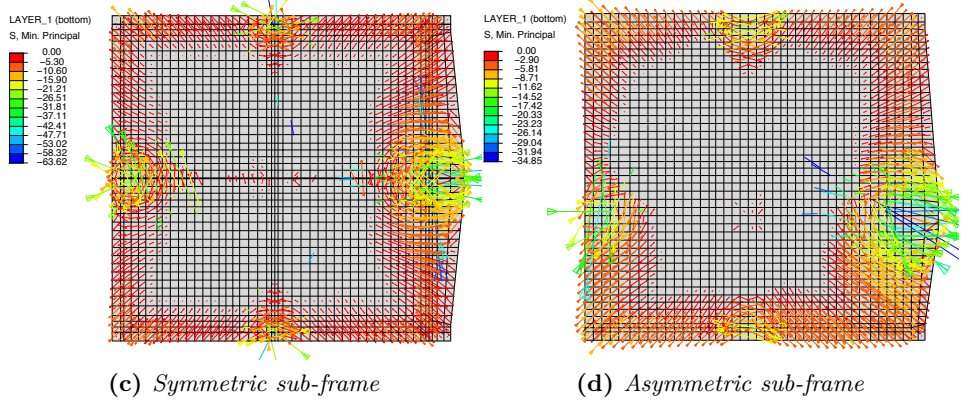


Figure 4.41: Tensile stresses in the bottom side of the slab (units in MPa)

Step 1



Step 2



Step 3

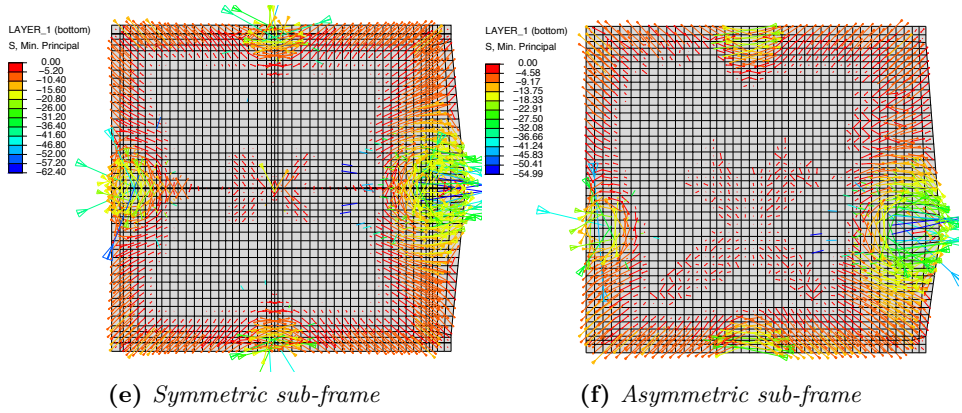


Figure 4.42: Compressive stresses in the bottom side of the slab (units in MPa)

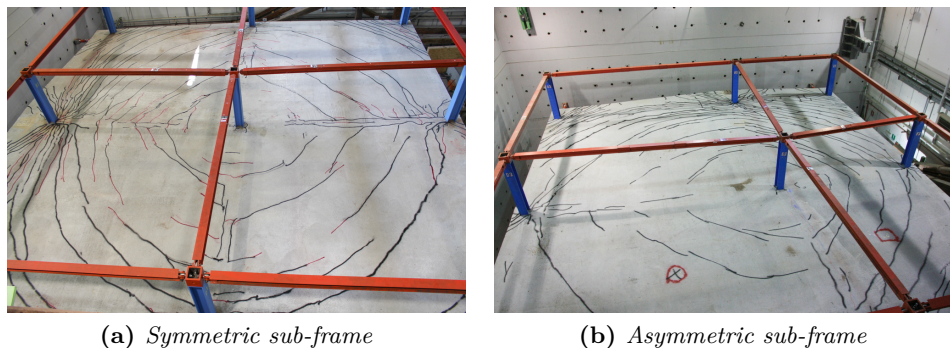


Figure 4.43: The cracking pattern on the top side of the slab

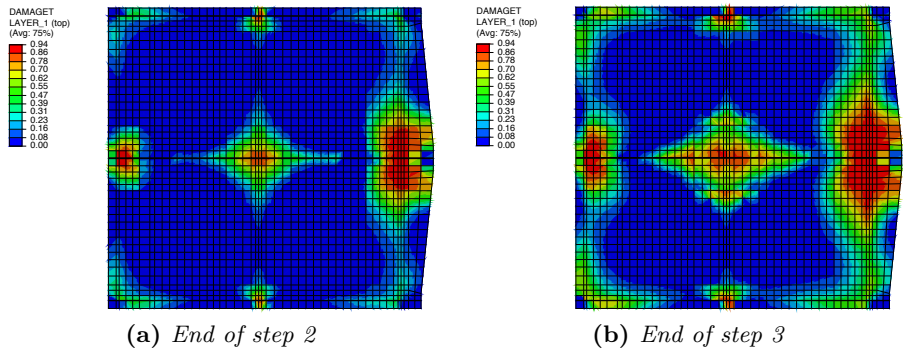


Figure 4.44: Tension damage pattern on the top side of the slab - symmetric sub-frame

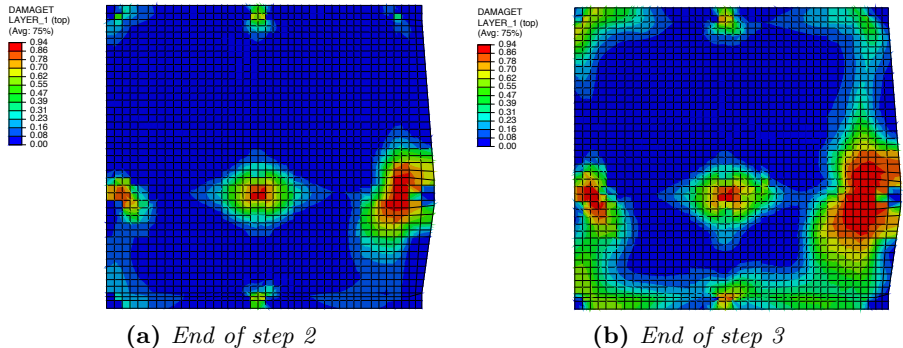
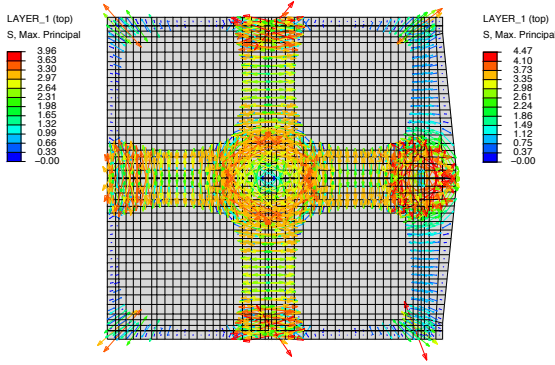
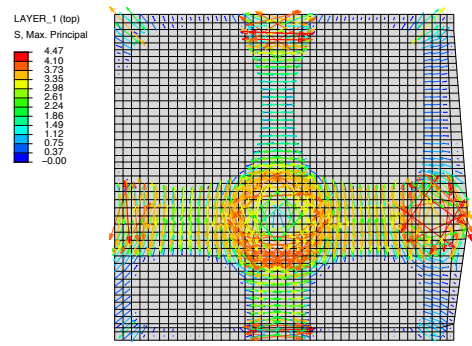


Figure 4.45: Damage pattern on the top side of the slab - asymmetric sub-frame

Step 1

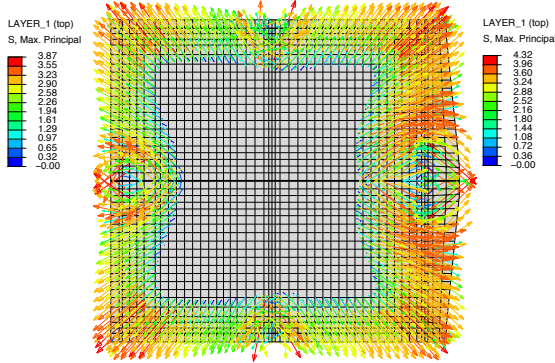


(a) *Symmetric sub-frame*

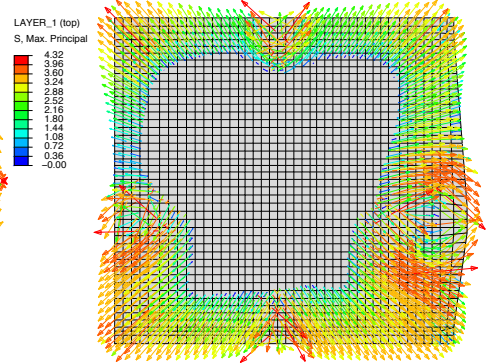


(b) *Asymmetric sub-frame*

Step 2

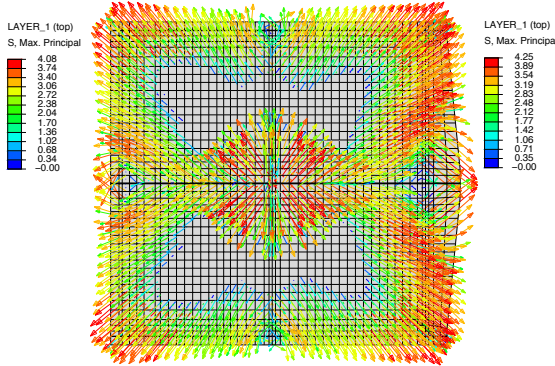


(c) *Symmetric sub-frame*

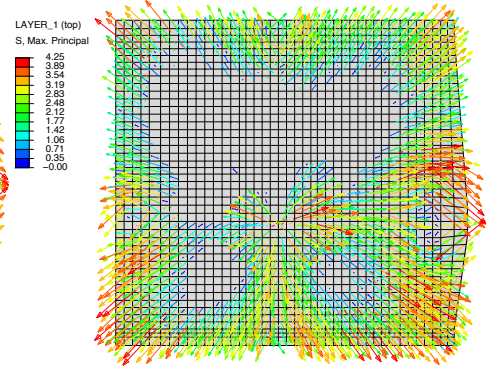


(d) *Asymmetric sub-frame*

Step 3



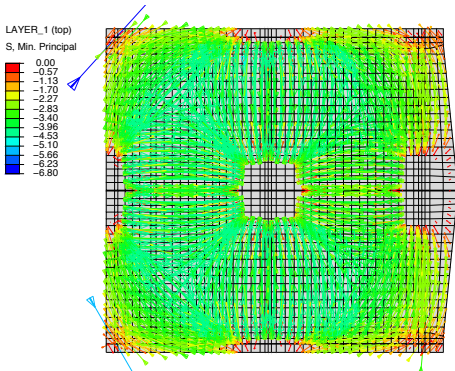
(e) *Symmetric sub-frame*



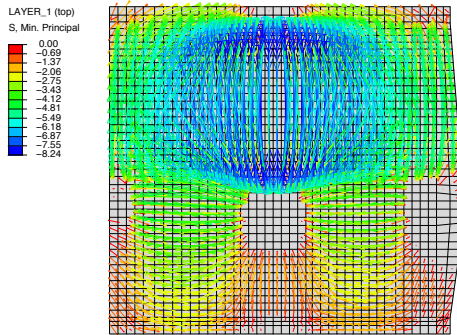
(f) *Asymmetric sub-frame*

Figure 4.46: Tensile stresses in the top side of the slab (units in MPa)

Step 1

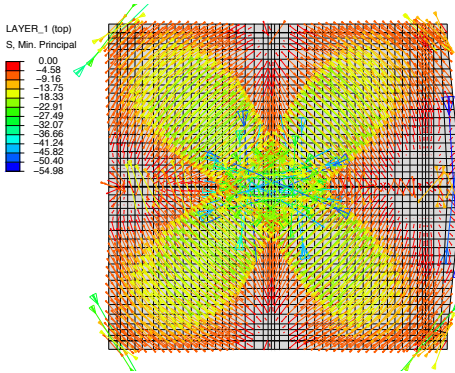


(a) *Symmetric sub-frame*

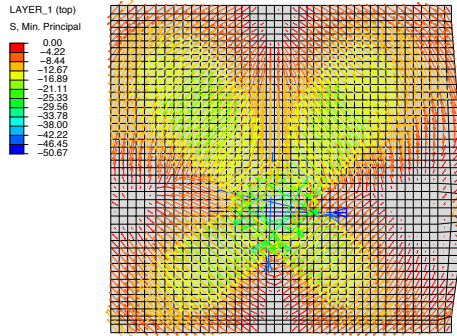


(b) *Asymmetric sub-frame*

Step 2

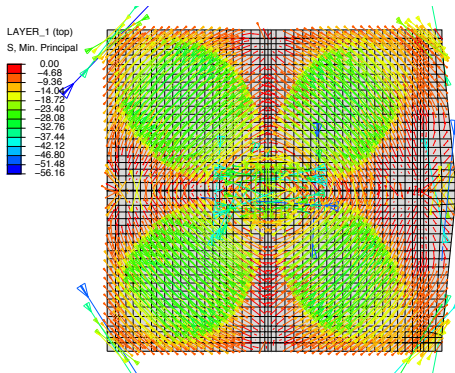


(c) *Symmetric sub-frame*

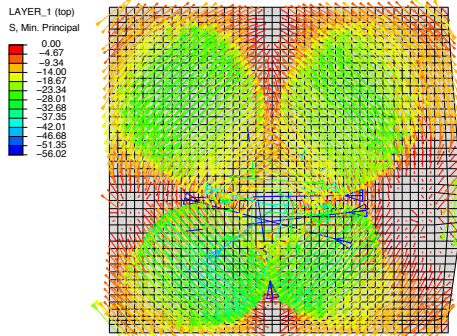


(d) *Asymmetric sub-frame*

Step 3



(e) *Symmetric sub-frame*



(f) *Asymmetric sub-frame*

Figure 4.47: Compressive stresses in the top side of the slab (units in MPa)

4.3.3 Concluding remarks

In this section, the finite element model of the sub-frame structures was described and the principal results compared to the experimental ones were reported. The numerical models consist of two different types of beam-to-column connections: pin connections and semi-rigid connections. This was to evaluate the best solution in terms of model complexity and computational burden that allows approximating the results with good accuracy. The numerical models were calibrated against the tests comparing the overall response of the systems and the results in the columns, joints, and slab. The model with semi-rigid connections is more accurate than the model with pin connections, even if placing ideal hinges at the intersection of the beams with the columns is an efficient way to model the frames and predicting the overall response and force distribution.

4.4 The contribution of the concrete slab

The concrete slab influences significantly the response of the frame structure. The load carrying capacity of the system can be enhanced if the slab allows developing alternative load paths and loads redistribution by the activation of membrane action.

In order to quantify the contribution of the concrete slab to the overall structural behaviour, a numerical analysis on the *symmetric* sub-frame without the slab was conducted, and the results compared with the ones carried out by the model previously described in which the slab is integrated in the system.

The geometrical properties of the frame and the material properties are described in §4.3.1. The only difference is in the slab modelling. The composite action between the steel beams and the concrete slab is ensured by the presence of a slab in correspondence of the beams as wide as the effective width suggested in the EN1994-1-1 [2004] in the sagging bending moment regions and equal to 1210 mm (see Figure 4.48). In order to reproduce the same loading condition of the model with the entire slab, the total load applied on the floor system (reported in Table 4.12) was divided by tributary area and applied on the slab of the current model.

The results of the analysis, reported in the following, focus on the overall response of the frame and of the beam-to-column connections. In particular, the following conclusions can be drawn:

- the load-deflection relationship of the central removed column, illustrated in Figure 4.49, shows a significant increase of the vertical displacement. The deflection at the end of the analysis is not compatible with the strength and ductility of the system, such as the joint rotation. The overall response of the frame gives an important un-

derstanding on the importance of the concrete slab to enhance the structural robustness;

- the joint rotations, reported in Figure 4.50, underline that the absence of the concrete slab causes a significant deformation in these structural members. The tests conducted on the full-framed structures tested at the University of Trento (§4.2), suggested that 70 mrad can be assumed as the maximum rotation of the beam-to column connections before the fracture of the bolts. The rotations up to about 190 mrad reached by the model without the slab are not admissible by the system.

4.4.1 Concluding remarks

In this section, a brief description of the finite element model, conducted on the *symmetric* sub-frame without the concrete slab, was presented with the main aim to demonstrate that the slab has an important contribution on the behaviour of the overall system and on the single structural member. The load-deflection response of the central column, as well as the joints rotation, obtained by the model without the slab, are significantly greater than the result of the model in which the entire slab is modelled. The concrete slab allows activating an alternative load path and force redistribution by means of membrane action. The ductility limit of the joints suggests that the frame will collapse before the column is completely removed. In this sense, it can be concluded that the concrete slab considerably improves the robustness of the structure in case of column loss and allows reducing the risk of progressive collapse.

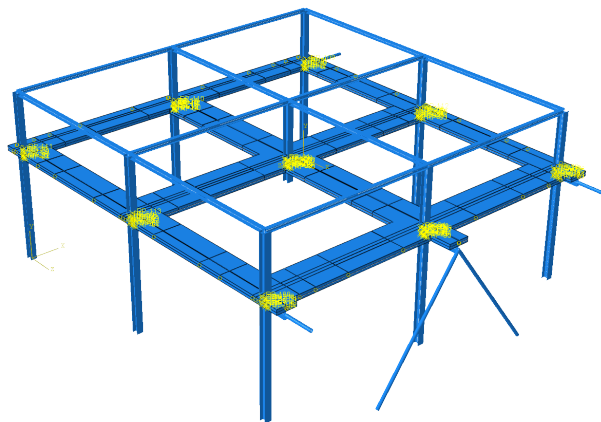


Figure 4.48: The finite element model on the symmetric sub-frame 'without' the concrete slab

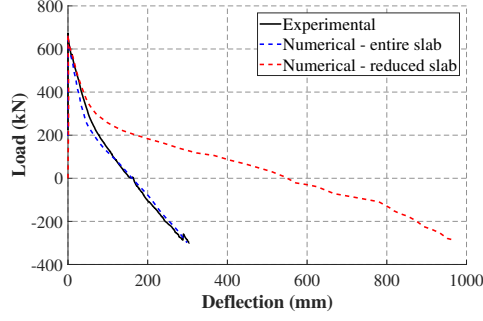


Figure 4.49: Load-deflection comparison of the models 'with' and 'without' the concrete slab

4.5 Modelling the full-frame structures

The comparison between experimental and numerical results in the sub-frames for the main parameters allowed the calibration of the finite element models and the definition of the numerical tools to develop numerical analyses of the full-frame structures (Figure 4.51).

The geometrical properties and dimensions of the frames are described in 4.2.1, while the material models and properties are reported in 4.3.1. As in the sub-frame models, the steel elements (beams, columns, and bracings) were modelled as 1D beam element (*BEAM 'B31'*) and the concrete slab as 2D shell element of approximately 250×250 mm side (*SHELL 'S4R'*). Cross sectional properties have been assigned to the structural elements as reported in Table 4.13. The only differences between the sub-frame and the full-frame structures are the presence of the X-bracing systems in both the plan directions (see Figure 4.1) and the rebars layout in the concrete slab. The slab was partitioned as in Figure 4.52 to consider the position of the reinforcement bars (Baldassino et al. [2018]). The cross sectional area and spacing of the rebars for each section of the slab are reported in Table 4.14 and Table 4.15 for the *symmetric* and *asymmetric* frame, respectively.

The complexity of the frames and the large number of beam-to-column connections required modelling the joints with a simple but efficient approach. The numerical analyses conducted for the sub-frame structures allowed concluding that the model with pin connections approximate with good accuracy the overall response of the system. The model with semi-rigid connections is more accurate, mainly for the behaviour of the joints, but more complex and computationally onerous. For these reasons, in the full-frame structures, the beams were connected to the columns with ideal hinges as shown in Figure 4.53. Furthermore, the model assumes rigid connections at the columns base and pin connections at the intersection of the X bracing systems.

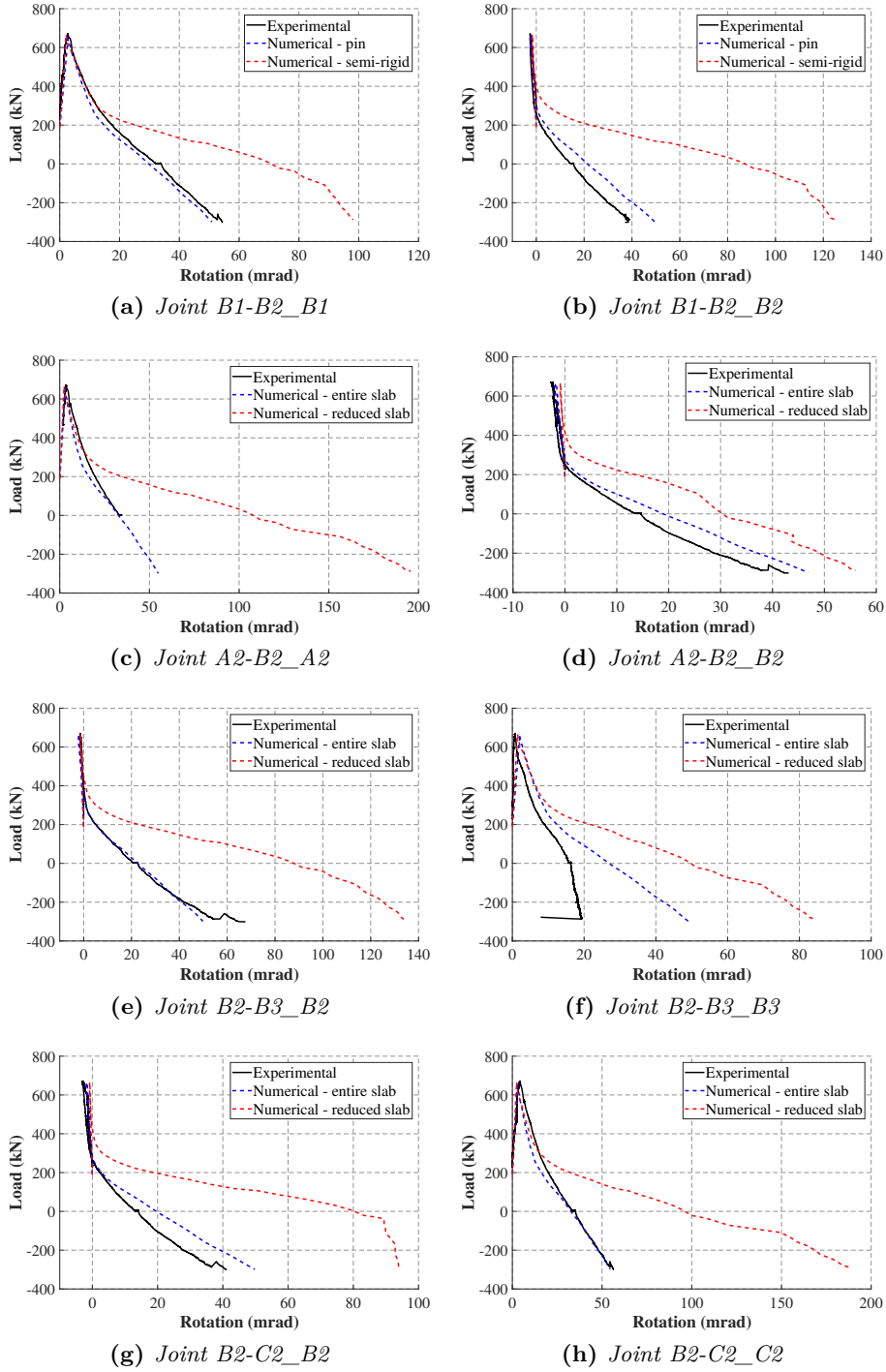


Figure 4.50: Joints rotation comparison of the models 'with' and 'without' the concrete slab

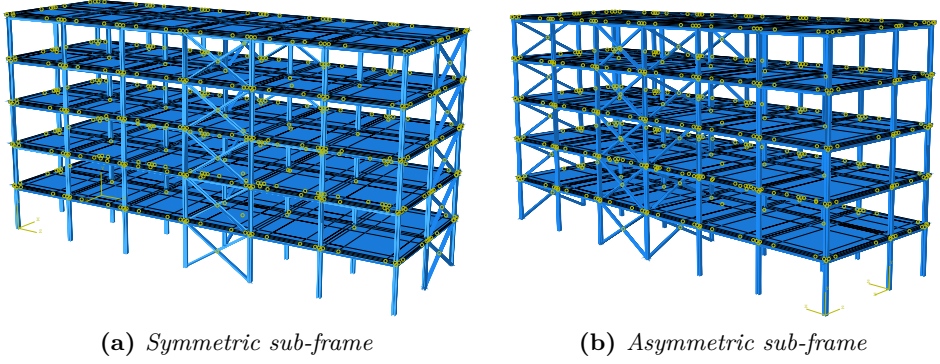


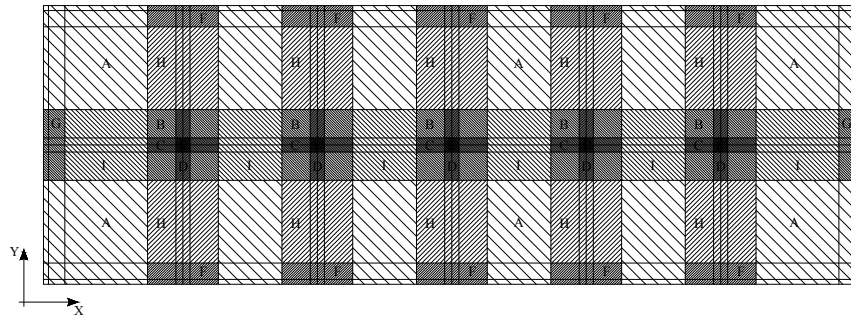
Figure 4.51: The finite element models on the full-frame

As in the sub-frames model, explicit analyses were conducted due to mechanical and geometrical non-linearities and to overcome convergence difficulties. In order to conduct a quasi-static analyses, the dynamic effects were neglected applying loads gradually with a smooth or linear law, and selecting a suitable 'time period' in the step definition. In the analysis, three loading steps were implemented as described in 4.3.1: i) in *step 1* (*time period* of 10) dead and live loads were applied and, simultaneously, the reaction force in the central column; ii) in *step 2* (*time period* of 5) column B2 removal was simulated (see Figure 4.1) applying an opposite reaction force; iii) during *step 3* (*time period* of 5) a tensile force was applied in the removed column and incremented up to obtain a vertical displacement of about 300 mm, as in the sub-frames and in the tests.

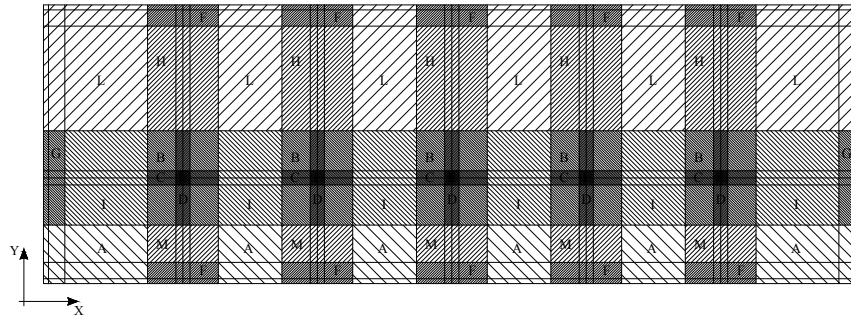
The loads applied on the frames are reported in Table 4.16. Live loads are in accordance with the prescription of the EN1991-1-1 [2004] for the Ultimate Limite State (ULS) in the office building category and described in the Baldassino et al. [2018].

Table 4.13: Cross-section of the structural elements

Structural element	Cross-section
Columns	HEB220
Beams	IPE240
Bracings (<i>X direction</i>)	L 120 × 120 × 10
Bracings (<i>Y direction</i>)	2L 120 × 120 × 10
Slab	165.0 mm



(a) *Symmetric sub-frame*



(b) *Asymmetric sub-frame*

Figure 4.52: Partition of the concrete slab - full-frame structures

Table 4.14: Rebars amounts in the slab sections - symmetric full-frame (units in mm)

Slab section	X direction Bottom	Y direction Bottom	X direction Top	Y direction Top
A	$\phi 10/150$	$\phi 10/150$	$\phi 10/150$	$\phi 10/150$
B	$\phi 10/150$	$\phi 10/150$	$\phi 10 + \phi 16/150$	$\phi 10 + \phi 16/150$
C	$2 \phi 10/150$	$\phi 10/150$	$\phi 10 + \phi 16/150$	$\phi 10 + \phi 16/150$
D	$\phi 10/150$	$2 \phi 10/150$	$\phi 10 + \phi 16/150$	$\phi 10 + \phi 16/150$
E	$2 \phi 10/150$	$2 \phi 10/150$	$\phi 10 + \phi 16/150$	$\phi 10 + \phi 16/150$
F	$\phi 10/150$	$\phi 10/150$	$\phi 10 + \phi 16/150$	$\phi 10/150$
G	$\phi 10/150$	$\phi 10/150$	$\phi 10/150$	$\phi 10 + \phi 16/150$
H	$\phi 10/150$	$\phi 10/150$	$2 \phi 10/150$	$\phi 10/150$
I	$\phi 10/150$	$\phi 10/150$	$\phi 10/150$	$2 \phi 10/150$

4.5.1 Comparison of the results between the sub-frames and the full-frames

The comparison of the analyses results between the sub-frame and full-frame, reported in this section, is important to understand whether the

Table 4.15: Rebars amounts in the slab sections - asymmetric full-frame (units in mm)

Slab section	X direction Bottom	Y direction Bottom	X direction Top	Y direction Top
A	$\phi 10/150$	$\phi 10/150$	$\phi 10/150$	$\phi 10/150$
B	$\phi 10/150$	$\phi 10/150$	$\phi 10 + \phi 16/150$	$\phi 10 + \phi 16/150$
C	2 $\phi 10/150$	$\phi 10/150$	$\phi 10 + \phi 16/150$	$\phi 10 + \phi 16/150$
D	$\phi 10/150$	2 $\phi 10/150$	$\phi 10 + \phi 16/150$	$\phi 10 + \phi 16/150$
E	2 $\phi 10/150$	2 $\phi 10/150$	$\phi 10 + \phi 16/150$	$\phi 10 + \phi 16/150$
F	$\phi 10/150$	$\phi 10/150$	$\phi 10 + \phi 16/150$	$\phi 10/150$
G	$\phi 10/150$	$\phi 10/150$	$\phi 10/150$	$\phi 10 + \phi 16/150$
H	$\phi 10/150$	2 $\phi 10/150$	2 $\phi 10/150$	$\phi 10/150$
I	$\phi 10/150$	$\phi 10/150$	$\phi 10/150$	2 $\phi 10/150$
L	2 $\phi 10/150$	2 $\phi 10/150$	$\phi 10/150$	$\phi 10/150$
M	$\phi 10/150$	$\phi 10/150$	2 $\phi 10/150$	$\phi 10/150$

Table 4.16: Loads applied on the full-frame structures

	Dead loads (kN/m ³)	Live loads (kN/m ²)
Steel density	78.50	-
Concrete density	24.75	-
Finishes, imposed load, movable partition	-	8.46

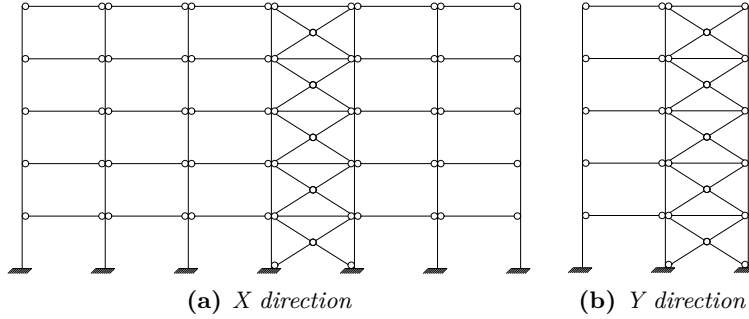


Figure 4.53: Beam-to-column connections in the full-frame structures

structural response of the sub-frames, subjected to column loss, is representative of the full-frame behaviour affected by the same event. The results focus on the load-deflection relation of the removed column, on the load redistribution among the columns, on the joint rotations and on the damage and stress patterns in the concrete slab. These quantities are assumed to approximate the overall structural response of the frames. The results will be reported for the three loading steps as described in 4.5, and are relative to a deflection of about 300 mm at the end of *step 3* in accordance to the

experimental outcomes.

As a first appraisal of the response, Figure 4.54 shows the load-deflection relationship of removed column B2. At the beginning of column removal, the stiffness of the *symmetric* and *asymmetric* frame is almost the same, both in the sub-frame and full-frame structures; at the end of the analyses, the *asymmetric* frame is more stiffer than the *symmetric* one. Furthermore, the vertical displacement in the sub-frames at the end of the column removal (*step 2*) is about twice the deflection in the full-frames. This allows concluding that the full-frame structures subjected to a central column loss exhibit limited deflections when the column is completely removed compared to the sub-frame cases. Consequently, during the application of the tensile force (*step 3*), the deflection increase in the full-frames is more marked than in the sub-frames. The ratio between the force at the end of *step 3* and the force acting before the column removal indicates that the full-frame structures can sustain larger forces and have more residual strength compared to the sub-frame structures. The effect of the asymmetry is also shown in Figures from 4.51 to 4.54 where the vertical displacements of the structures at the end of the three loading steps are represented.

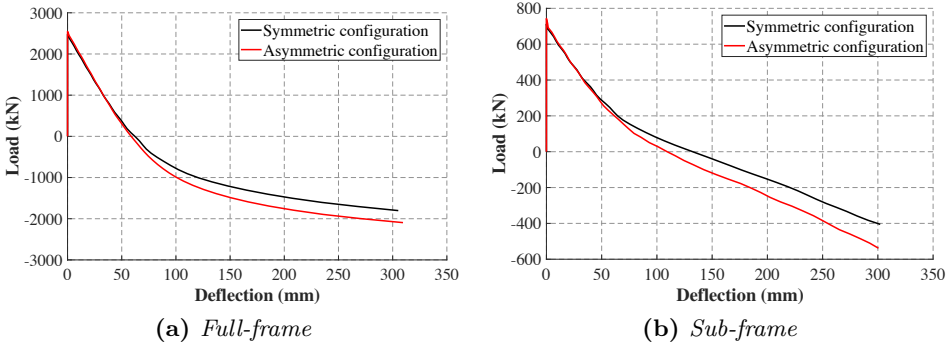


Figure 4.54: Load-deflection relationship of the removed column B2

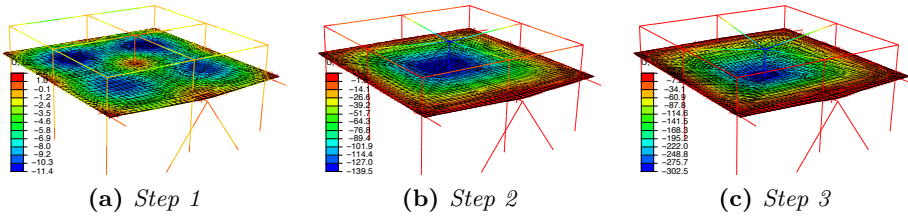


Figure 4.55: Vertical displacements of the sub-frame - symmetric configuration

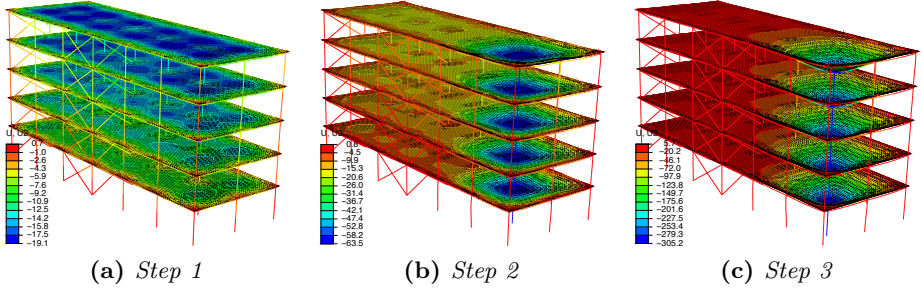


Figure 4.56: Vertical displacements of the full-frame - symmetric configuration

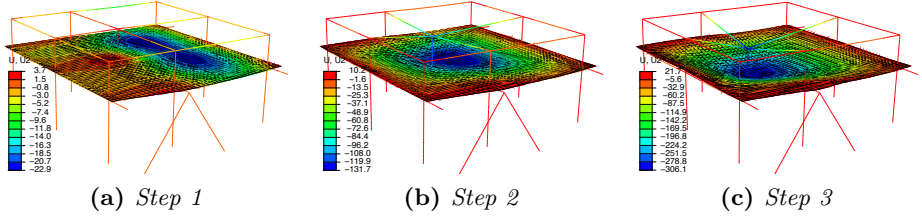


Figure 4.57: Vertical displacements of the sub-frame - asymmetric configuration

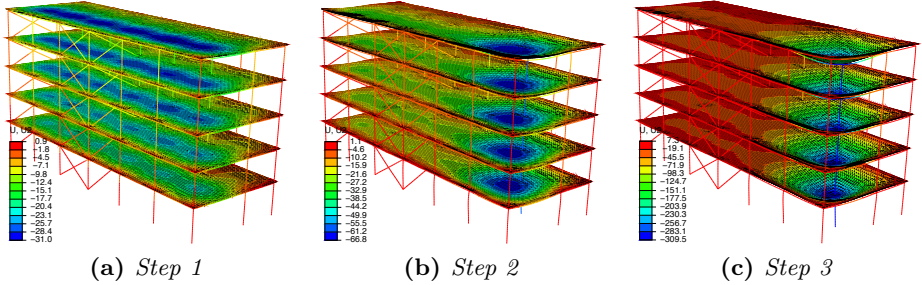


Figure 4.58: Vertical displacements of the full-frame - asymmetric configuration

Thrusts in the columns

Load redistribution in the structure after column loss can be evaluated by the analysis of the axial forces in the columns (named as in Figure 4.59). The following figures (Figures 4.60 to 4.64) report the axial force in the columns for each floor of the *symmetric* and *asymmetric* full-frame structures, while Figure 4.65 is relative to the sub-frame case. Furthermore, the difference in percentage is reported with respect to the value at the end of *step 1* (reported in Table 4.17). A positive percentage means that the force increases, while a negative one indicates a decrease.

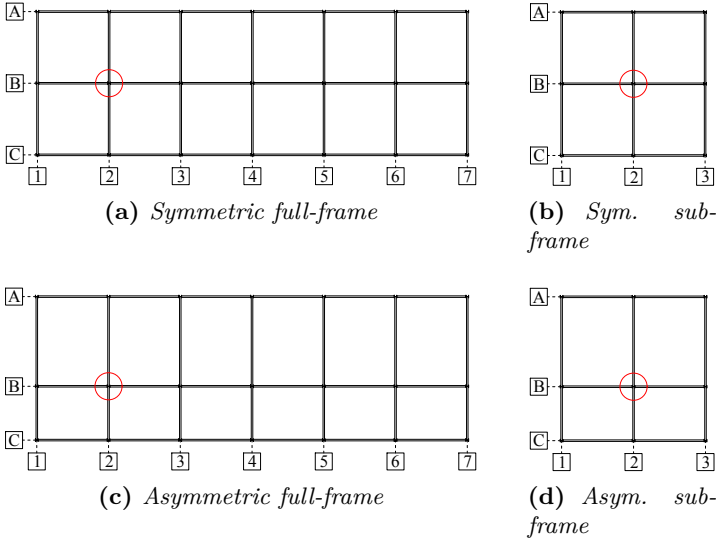


Figure 4.59: Nomenclature of the columns

As a first appraisal of the graphs, it is important to notice that a column is not influenced by the removal of column B2 if the three symbols that indicate the force value for each step of the analyses, are almost coincident. On the contrary, scattered symbols indicate that the column is affected by the event. Both in the *symmetric* and *asymmetric* full-frame, the columns in the transverse frame '1', '2' and '3' are directly influenced by the removal of column B2. The variation in the remaining frames '4' to '7' is not relevant. The effects of the asymmetry is underlined by the evolution of the axial force in column C2 (see 'Frame C' in Figure 4.60): after the column loss, the force increases by about 170% in the *asymmetric* frame, while the increase in the *symmetric* one is just of about 67%. The variations in the upper floors are similar to the values obtained for the first floor, concluding that the first floor is representative of the response of the overall building. The comparison between the full-frame and the sub-frame structures (Figure 4.60 and Figure 4.65, respectively) clearly indicate that the variations in the most affected columns (A2, B1, B3 and C2) in the sub-frame is about twice the variation in same columns in the full-frame case. The corner columns in the sub-frames sustained a decrease of the axial force after the column loss, while an increase can be seen in the full-frame structures.

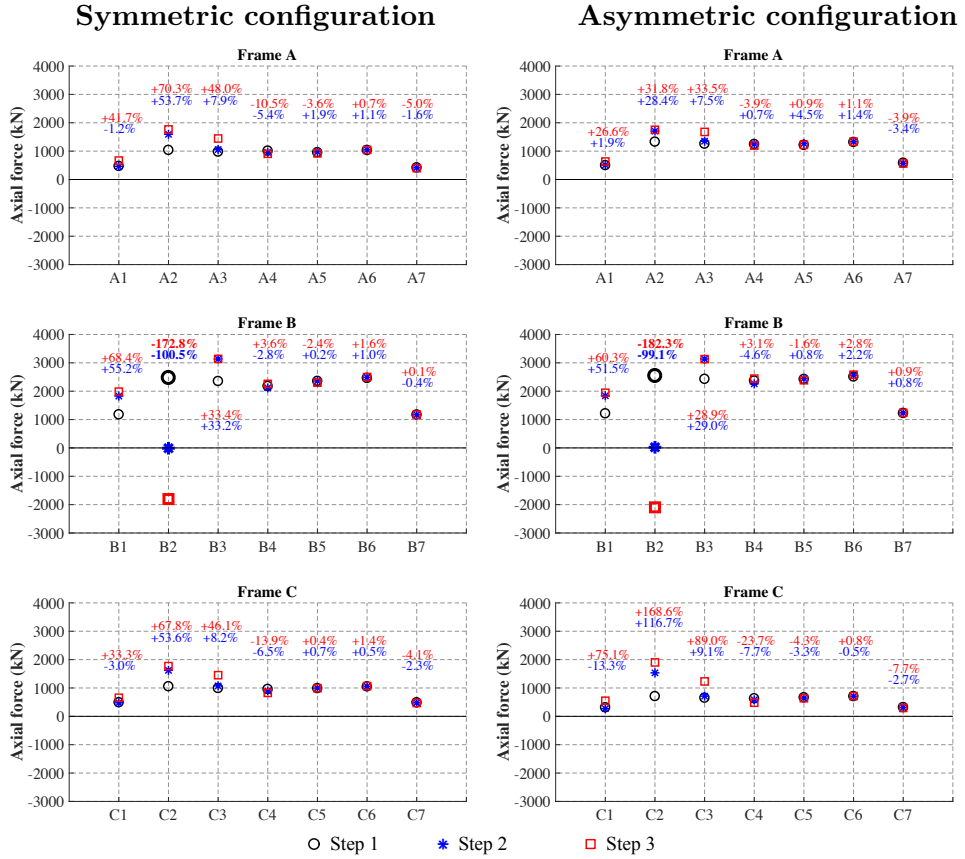


Figure 4.60: Axial force in the columns - full-frame - floor 1

Table 4.17: Axial force in the columns at the end of step 1 (units in kN)

Column	Symmetric configuration				
	Floor 1	Floor 2	Floor 3	Floor 4	Floor 5
A1	471.5	379.9	285.7	190.1	93.0
A2	1036.8	833.0	626.1	416.2	206.5
A3	976.4	788.8	594.5	396.3	196.0
A4	1011.9	859.8	655.5	446.2	233.3
A5	956.1	772.3	581.6	389.1	192.4
A6	1035.3	833.1	627.2	417.7	207.2
A7	420.2	358.4	276.0	191.9	105.7
B1	1177.8	943.6	709.0	473.8	237.6
B2	2476.8	1958.3	1459.3	971.6	493.1
B3	2353.2	1872.3	1400.9	934.4	471.9
B4	2178.4	1744.7	1303.6	881.6	475.9
B5	2357.2	1874.8	1402.6	935.2	472.3
B6	2461.6	1946.6	1450.9	966.0	490.5
B7	1171.6	954.4	718.2	482.2	250.3
C1	491.8	397.8	299.6	198.8	96.5
C2	1056.1	849.4	638.8	426.6	212.3
C3	991.9	803.2	607.1	405.8	199.7
C4	959.5	778.9	589.7	394.5	193.1
C5	988.9	801.7	603.9	401.4	195.4
C6	1051.2	846.0	636.6	425.8	212.4
C7	487.9	395.8	298.6	198.8	96.7

Column	Asymmetric configuration				
	Floor 1	Floor 2	Floor 3	Floor 4	Floor 5
A1	501.4	501.4	378.1	251.3	125.6
A2	1329.5	1064.3	799.6	533.3	266.2
A3	1256.7	1012.7	762.6	510.9	252.9
A4	1245.0	1060.5	817.3	561.6	307.9
A5	1214.2	980.7	739.0	492.1	243.9
A6	1317.0	1055.9	791.8	525.5	260.3
A7	586.9	499.0	381.9	267.6	152.2
B1	1213.5	969.8	728.0	485.3	245.0
B2	2545.7	2006.0	1492.7	991.7	505.6
B3	2429.0	1927.3	1440.3	959.6	486.4
B4	2369.2	1900.6	1419.3	962.6	527.2
B5	2425.5	1923.3	1436.5	956.5	484.0
B6	2511.5	1979.5	1472.1	978.7	497.2
B7	1227.0	1000.1	751.9	510.1	275.8
C1	314.0	256.4	193.9	129.3	60.1
C2	707.8	580.1	440.4	293.9	144.3
C3	651.7	535.5	407.8	272.4	132.3
C4	630.6	522.3	399.4	269.0	129.9
C5	666.3	545.4	411.4	273.0	129.4
C6	707.1	577.8	439.5	293.9	144.4
C7	320.0	263.6	200.4	133.1	63.6

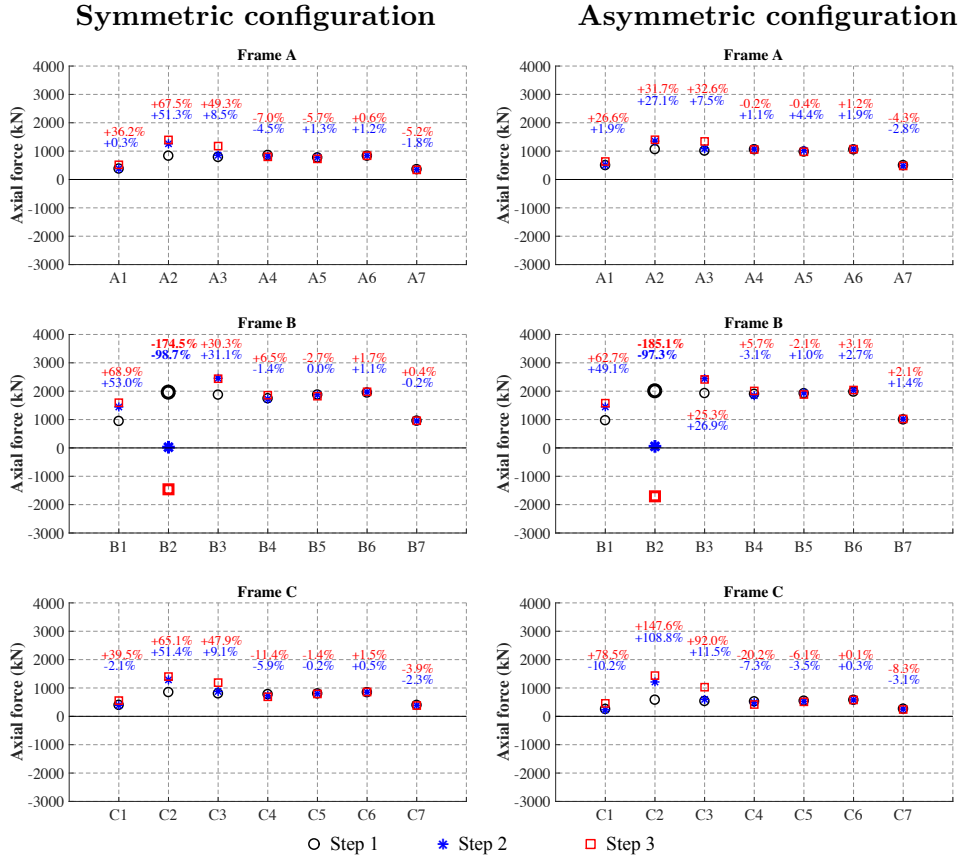


Figure 4.61: Axial force in the columns - full-frame - floor 2

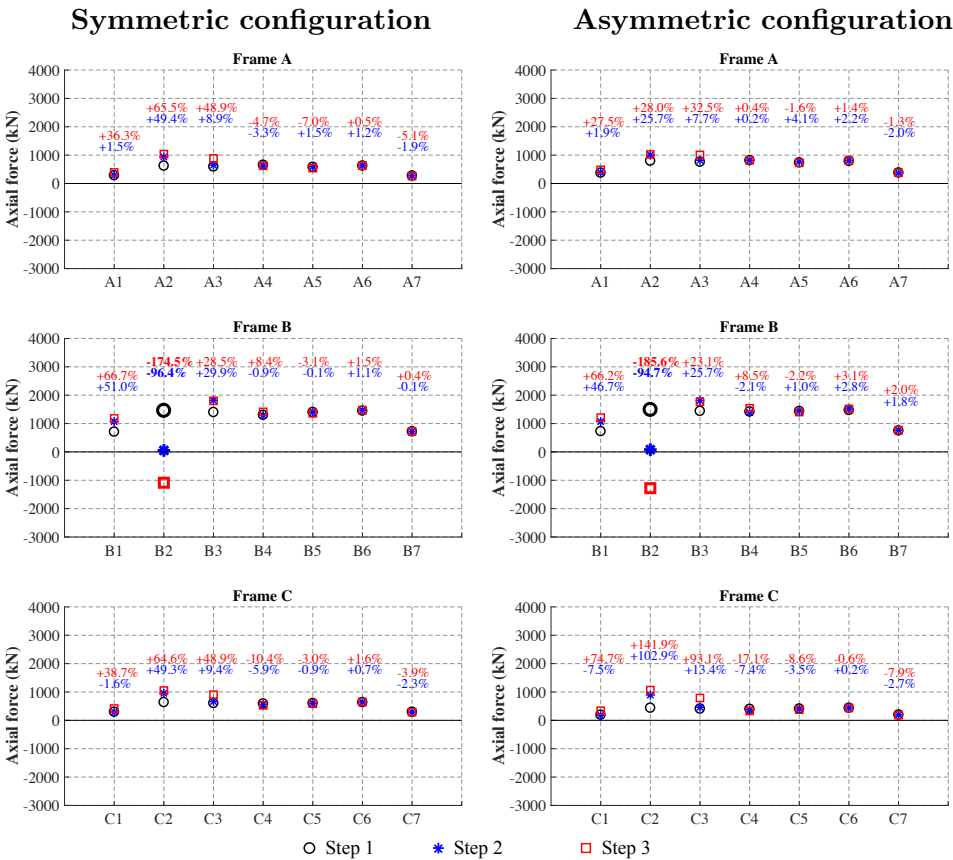


Figure 4.62: Axial force in the columns - full-frame - floor 3

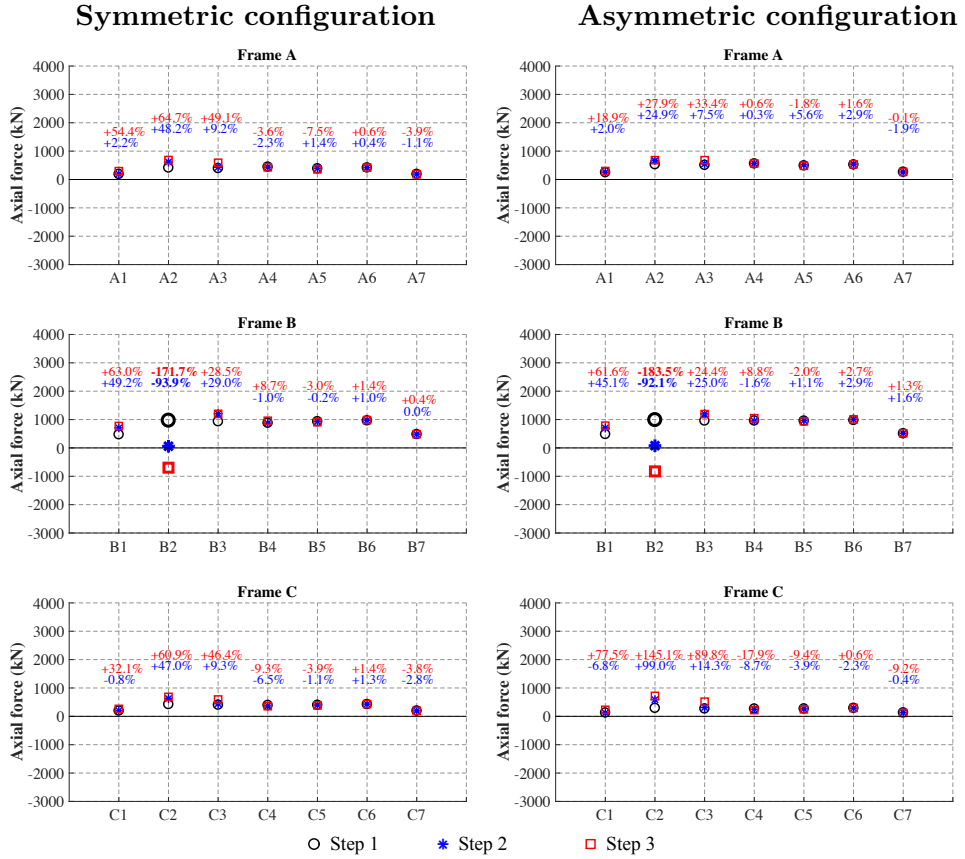


Figure 4.63: Axial force in the columns - full-frame - floor 4

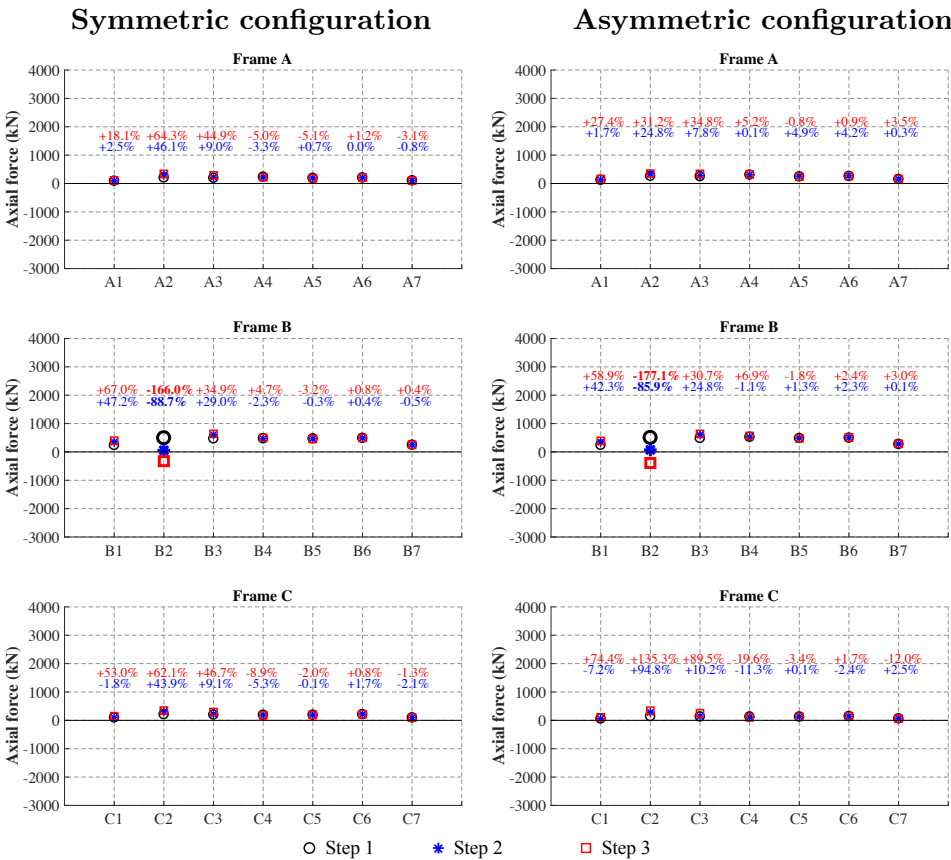


Figure 4.64: Axial force in the columns - full-frame - floor 5

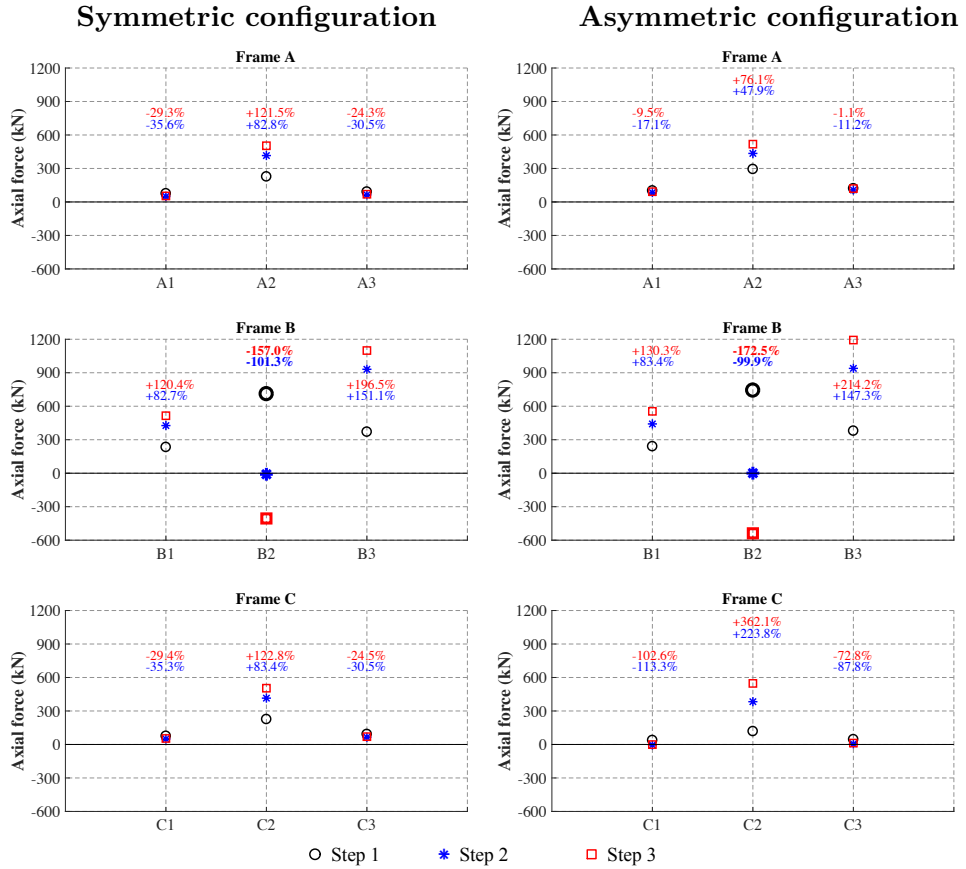


Figure 4.65: Axial force in the columns - sub-frames

Actions in the joints

The 'collapse' of the frames tested in the laboratory (described in §4.2) was achieved when the bolts in the bottom row in a joint fractured. Joint rotation is an important parameter to evaluate the ductility of the structures and to determine the reserve of strength associated to the column loss scenario. Figure 4.66 reports the rotation in the most important joints of the sub-frame structures, while Figure 4.67 and Figure 4.68 relate to the full-frame, for the *symmetric* and *asymmetric* configuration, respectively. In particular, figures (a) illustrate the rotation values at the end of the loading steps, while figures (b) report the variation in percentage in *step 2* and *3* with respect to floor 1.

The rotations in the joints of the first floor of the overall building are representative of the upper floors, because similar values can be measured along the storeys. Even if relevant variations are reported in the figures, it is important to note that, in many cases, the rotation at the end of *step 1* is 'small', and thus the variation is negligible.

Modelling the sub-frame structure has the consequence of incrementing the rotation in the joints that connect the 'central' beams to the external columns. The rotation in these joints in the full-frames are smaller compared to the sub-frame cases because of higher restraints provided by the columns. Similar conclusions can be drawn by the analyses of the rotation in the joints between the central beams to the removed column (B2), where modelling the sub-frames has the consequence of incrementing the rotation.

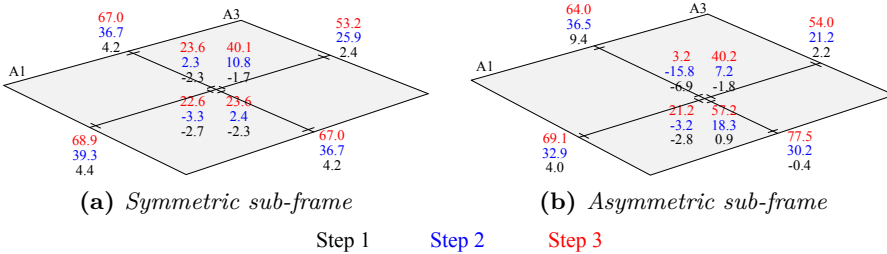


Figure 4.66: Joint rotations - full-frame structures (units in mrad)

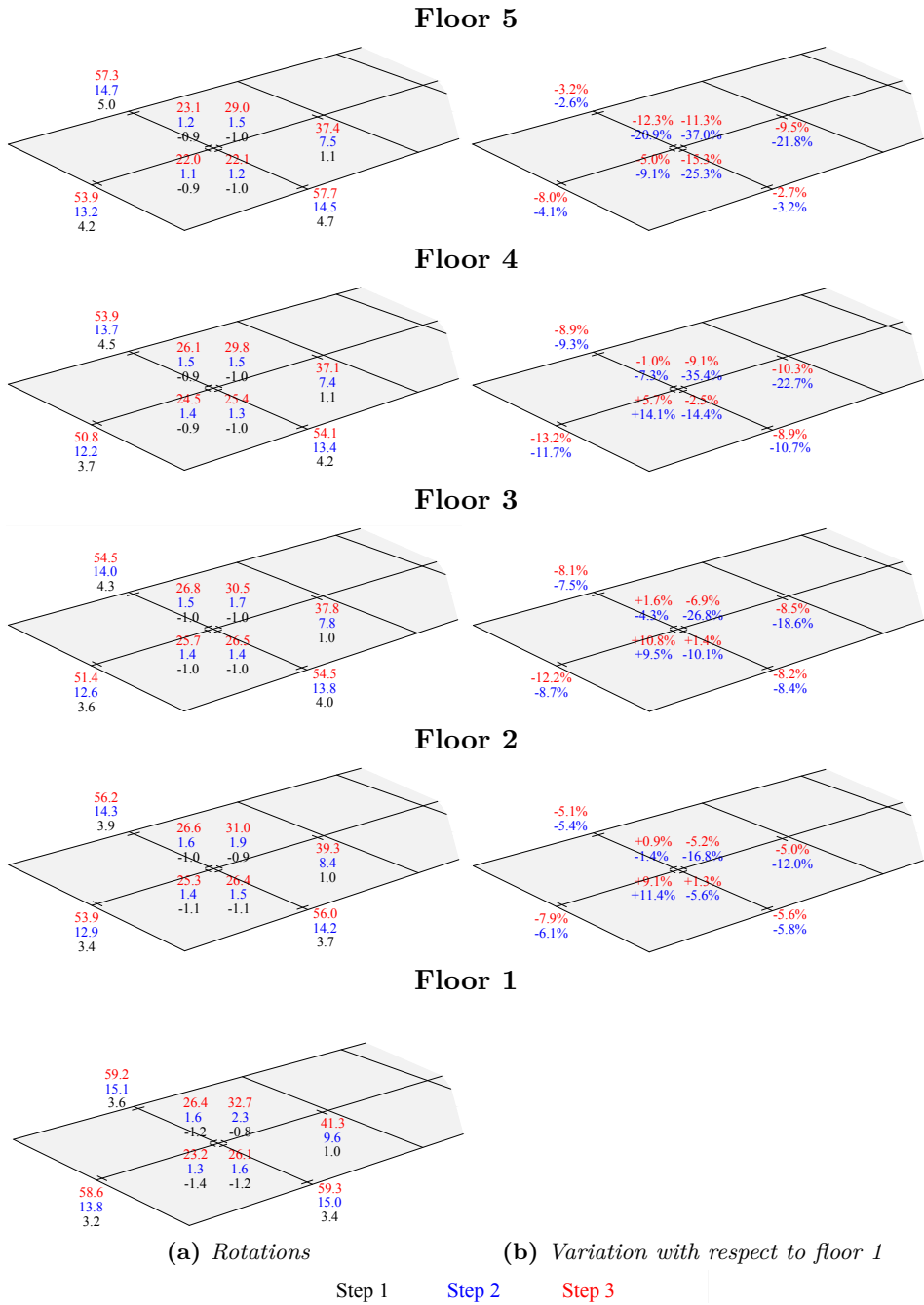


Figure 4.67: Joint rotations - symmetric full-frame structure (units in mrad)

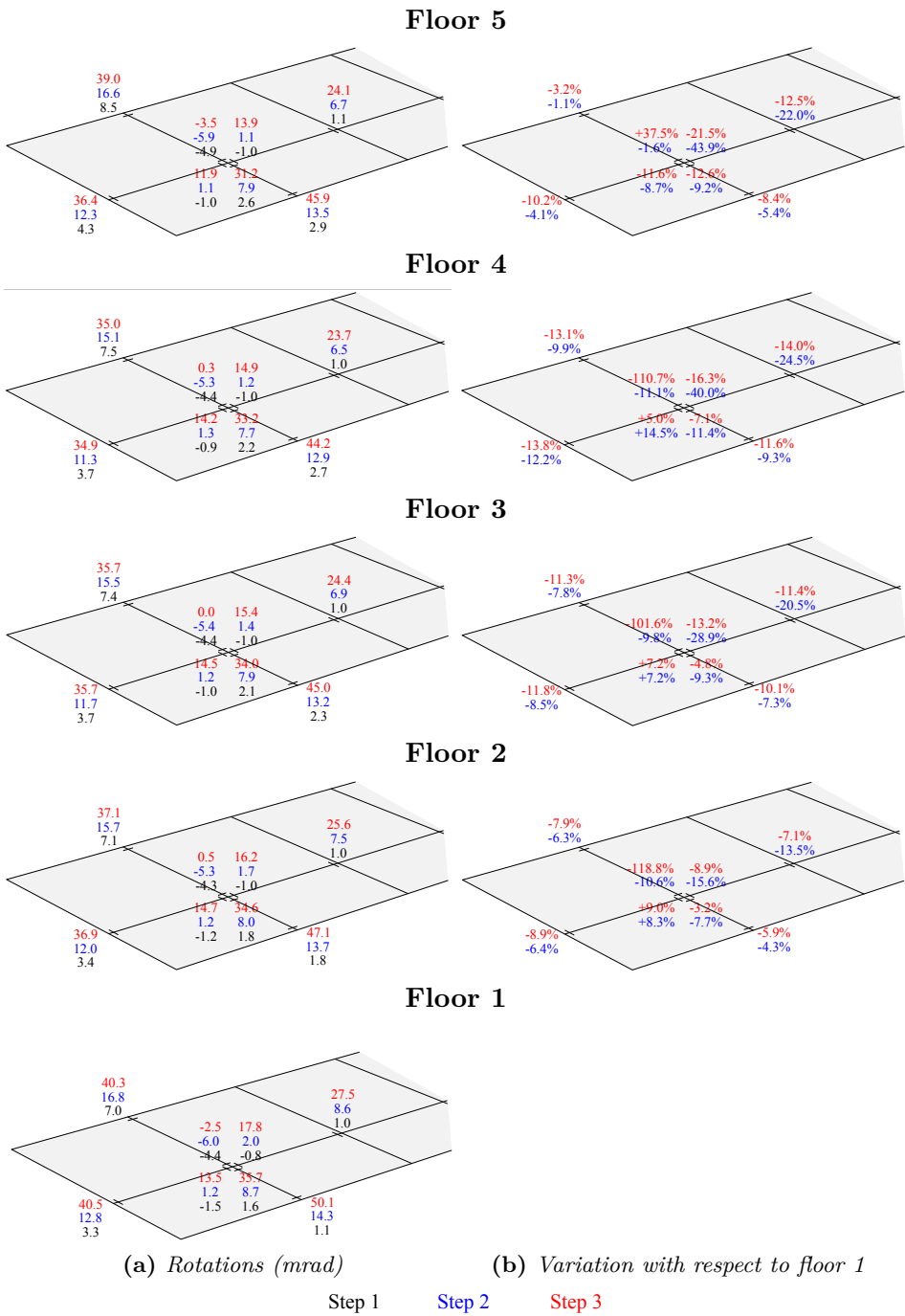


Figure 4.68: Joint rotations - asymmetric full-frame structure (units in mrad)

The slab

The tension damage pattern in the concrete slab is illustrated in Figure 4.69 and Figure 4.70 for the bottom and top side, respectively. Figure 4.71 and Figure 4.72 relate to the *asymmetric* frame. As in the sub-frames (see §4.3.2), the spread of the damage from *step 2* to *step 3* is evident. The cracked area at the end of the column removal decreases from the floor 1 to floor 5, because of force transmission and loads redistribution among the entire structure, as illustrated in Figure 4.69 (a) and Figure 4.71 (a). Furthermore, any relevant variations of the damage pattern in the upper side of the slab along the height of the building are shown (Figure 4.70 (b) and Figure 4.72 (b)).

The next figures (4.69 to 4.84) show the comparison of the stress distribution in the slab between the full-frame and the sub-frame structures. In particular, Figure 4.73 and Figure 4.74 relate to the tensile stresses in the bottom side of the slab, for the symmetric full-frame and sub-frame, respectively. At the end of *step 1*, the evolution of the stresses is similar and the symmetry of the frames is respected. At the end of *step 2*, the development of the yield lines across the diagonal of the slab panels affected by column collapse is evident, even if the slab in the sub-frame (Figure 4.74 (b)) is more cracked compared to the full-frame case. At the end of *step 3*, the regions close to the yield lines are completely cracked, because tensile membrane action starts to develop. Figure 4.73 shows how the removal of column B2 affects the symmetric left side of the entire structure, while the right side does not change the stress distribution acting before the column loss. The evolution of the tensile stresses in the bottom side of the *asymmetric* slab, shown in Figure 4.75 and Figure 4.76, is similar to the *symmetric* case. Focusing on the top side of the slab, the tensile stress distribution in the *symmetric* structures, shown in Figure 4.77 and Figure 4.78 for the full-frame and sub-frame, respectively, underline the regions where hogging bending moment acts, in particular close to the supports and along the perimeter of the slab panels affected by the column removal. At the end of *step 3*, tensile stresses in the area close to the removed column B2 in the sub-frame (Figure 4.78 (c)) start to develop, contrary to what happens in the full-frame (Figure 4.77). Any substantial difference in the *asymmetric* frames are pointed out as shown in Figure 4.79 and Figure 4.80.

The compressive stress distributions in the bottom side of the slab are illustrated in Figure 4.81 and Figure 4.82 for the *symmetric* full and sub-frames, respectively, and in Figure 4.83 and Figure 4.84 for the *asymmetric* ones. Figures 4.81 to 4.84 refer to the compressive stresses in the top side of the slab. It is interesting to note that the stresses in the top side are concentric with respect to column B2 as shown at the end of the test in Figure 4.43. The comparison between the full-frame and the sub-frame structures highlight a similar behaviour of the slab, both in the bottom and top sides, with

a particular focus on the developing of tensile and compressive membrane action at the end of *step 3*.

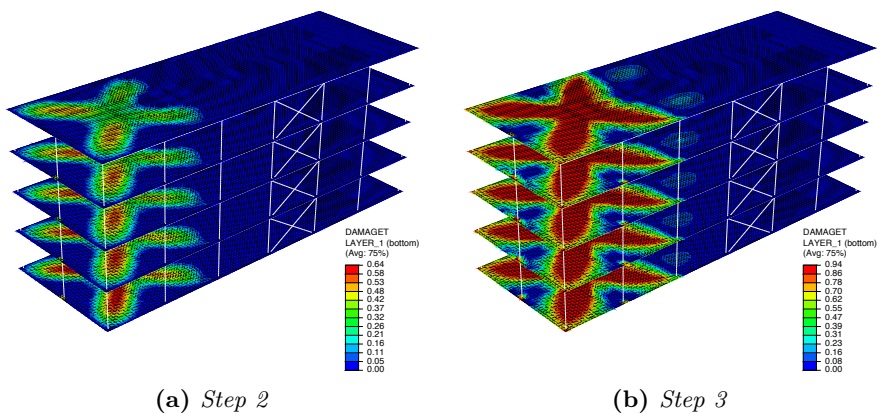


Figure 4.69: Tension damage pattern in the bottom side of the slab - symmetric full-frame

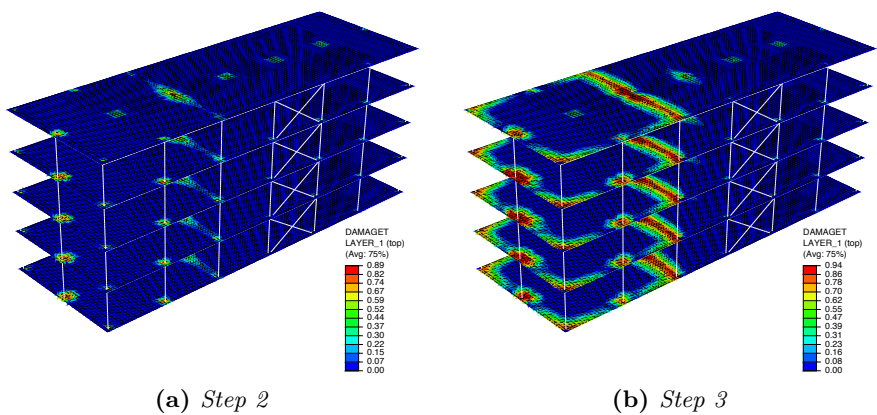


Figure 4.70: Tension damage pattern in the top side of the slab - symmetric full-frame

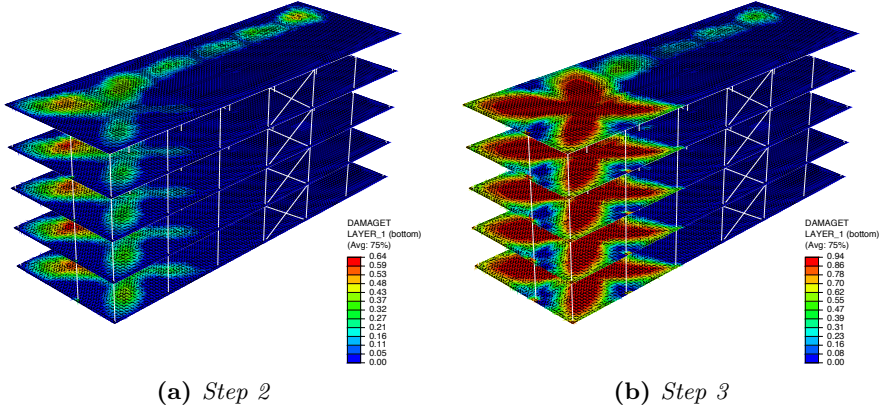


Figure 4.71: Tension damage pattern in the bottom side of the slab - asymmetric full-frame

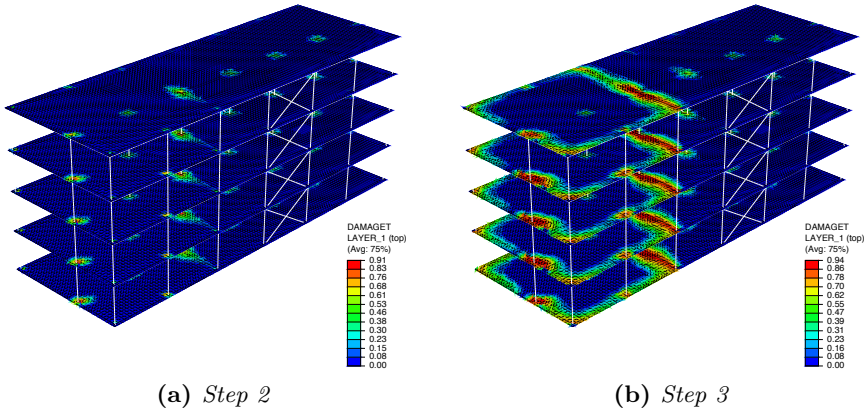


Figure 4.72: Tension damage pattern in the top side of the slab - asymmetric full-frame

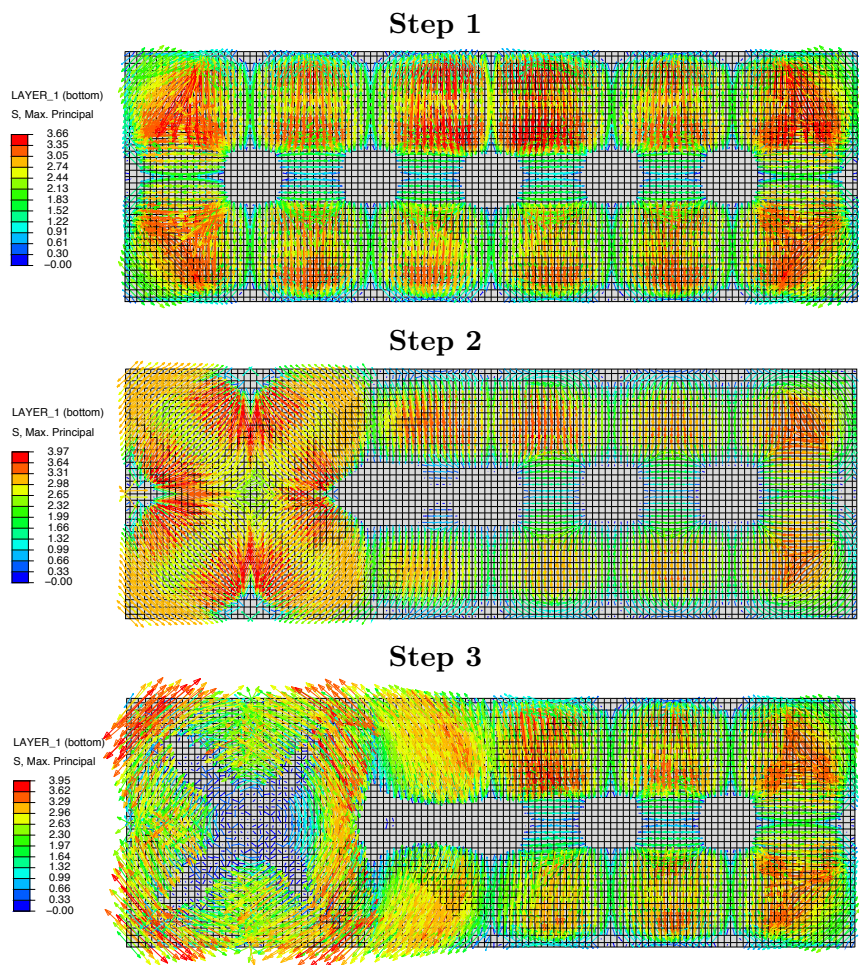


Figure 4.73: Tensile stresses in the bottom side of the slab - symmetric full-frame (units in MPa)

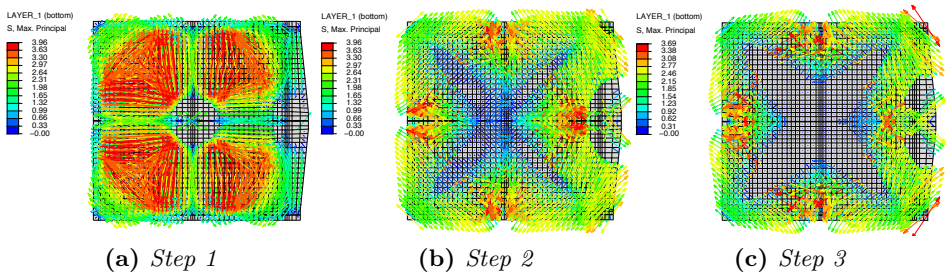


Figure 4.74: Tensile stresses in the bottom side of the slab - symmetric sub-frame (units in MPa)

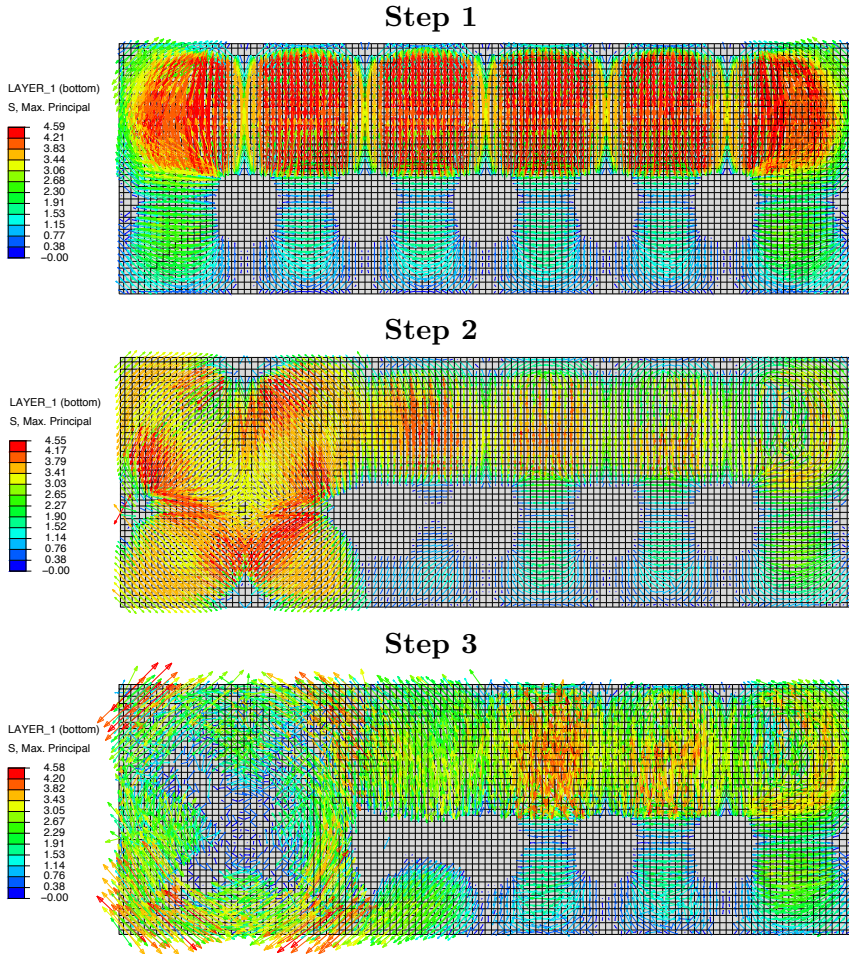


Figure 4.75: Tensile stresses in the bottom side of the slab - asymmetric full-frame (units in MPa)

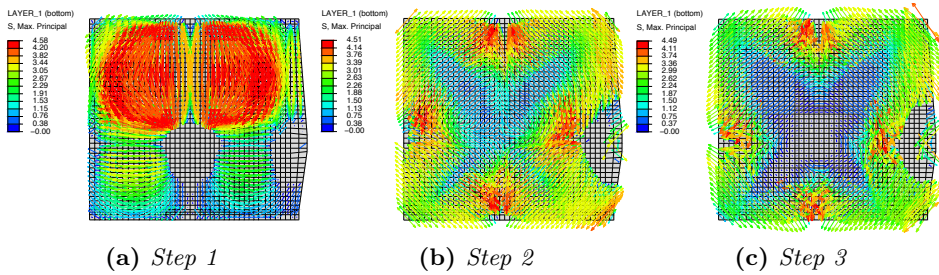


Figure 4.76: Tensile stresses in the bottom side of the slab - asymmetric sub-frame (units in MPa)

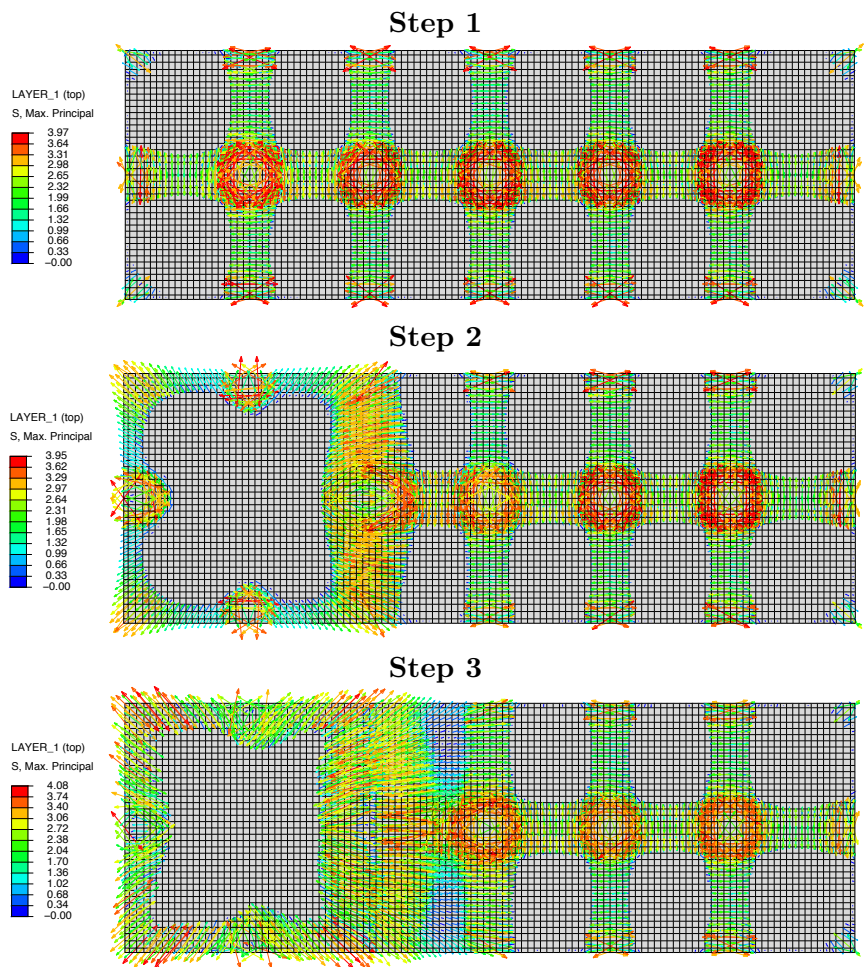


Figure 4.77: Tensile stresses in the top side of the slab - symmetric full-frame (units in MPa)

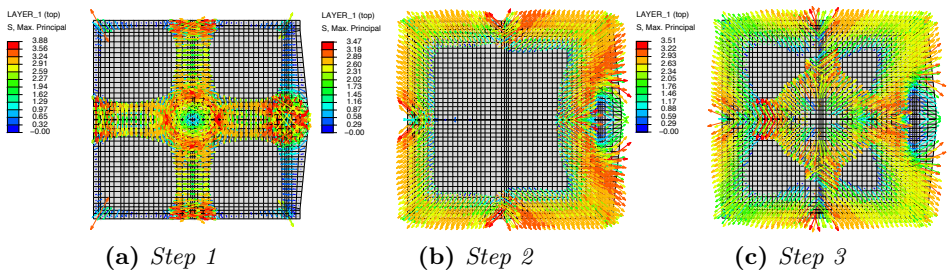


Figure 4.78: Tensile stresses in the top side of the slab - symmetric sub-frame (units in MPa)

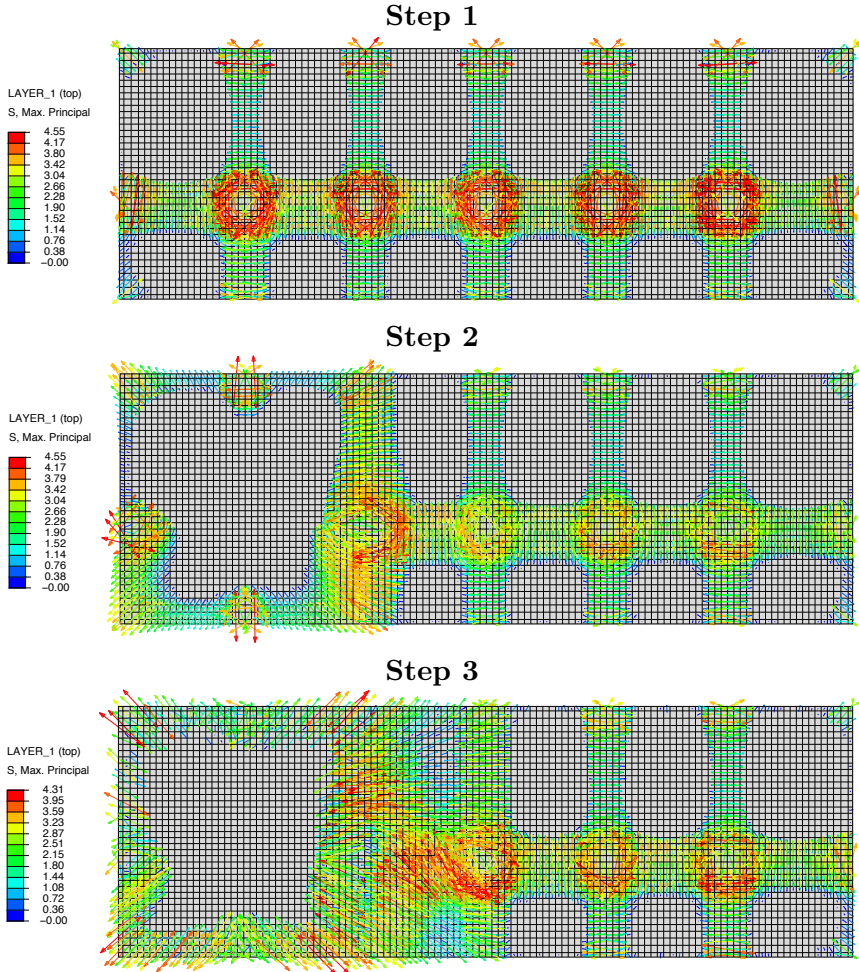


Figure 4.79: Tensile stresses in the top side of the slab - asymmetric full-frame (units in MPa)

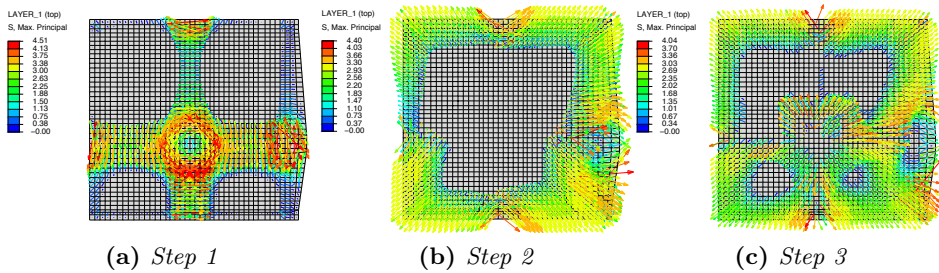


Figure 4.80: Tensile stresses in the top side of the slab - asymmetric sub-frame (units in MPa)

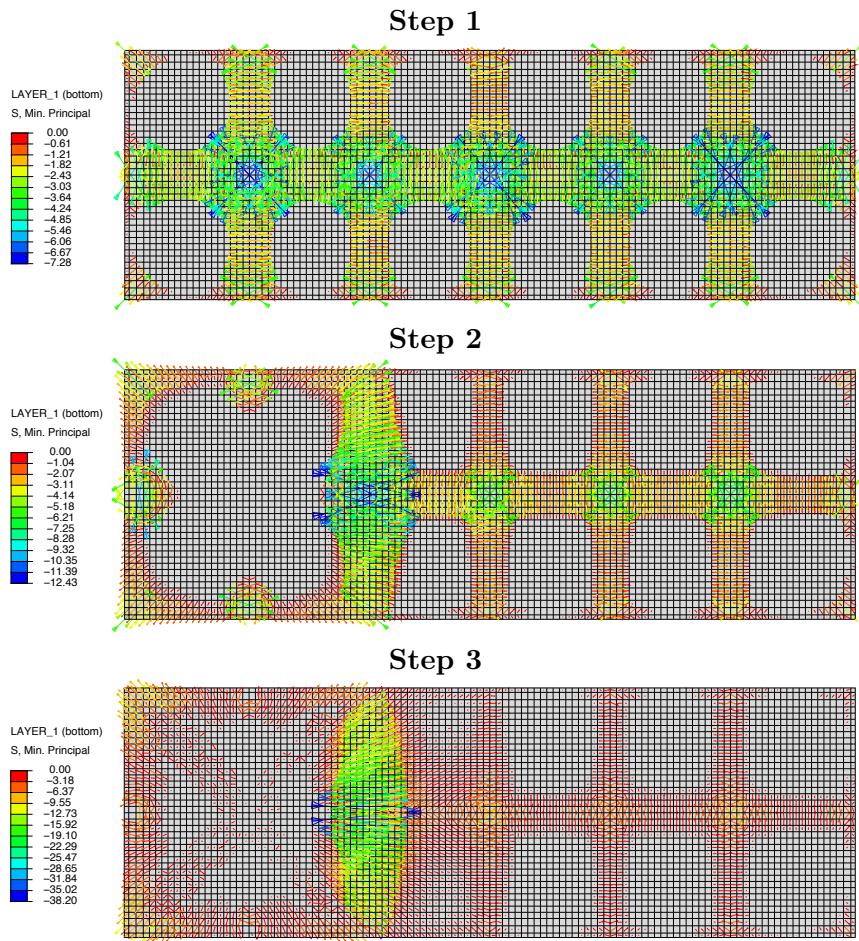


Figure 4.81: Compressive stresses in the bottom side of the slab - symmetric full-frame (units in MPa)

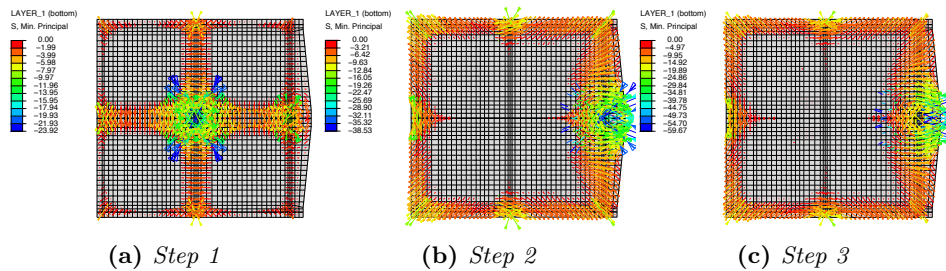


Figure 4.82: Compressive stresses in the bottom side of the slab - symmetric sub-frame (units in MPa)

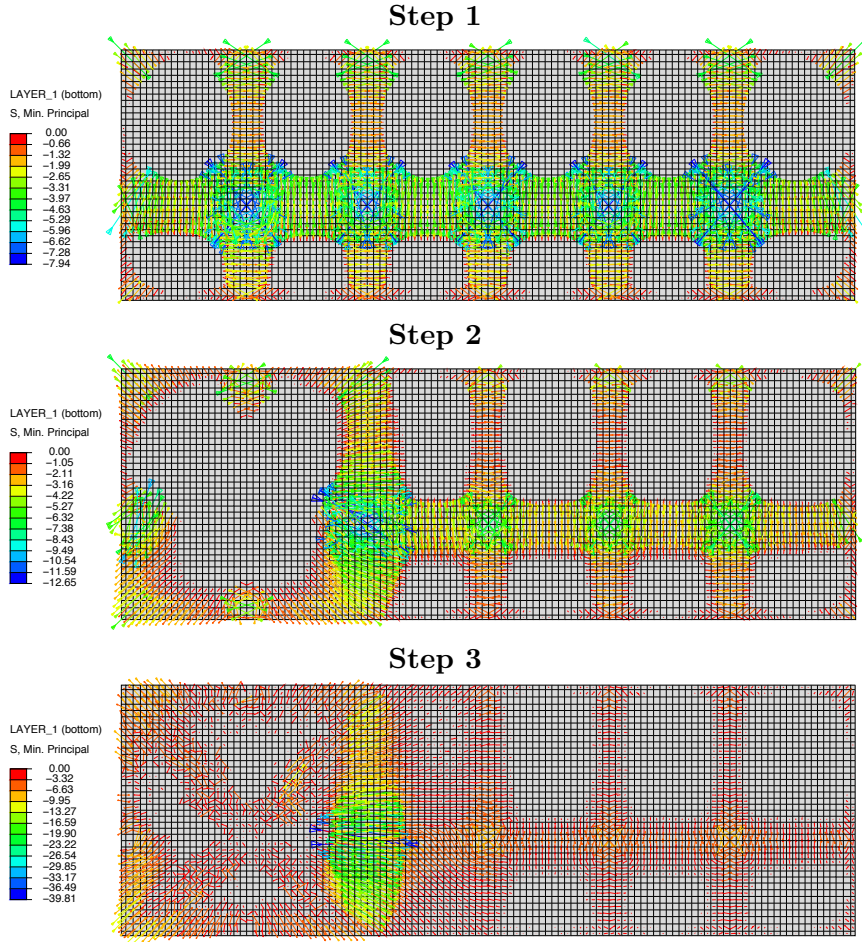


Figure 4.83: Compressive stresses in the bottom side of the slab - asymmetric full-frame (units in MPa)

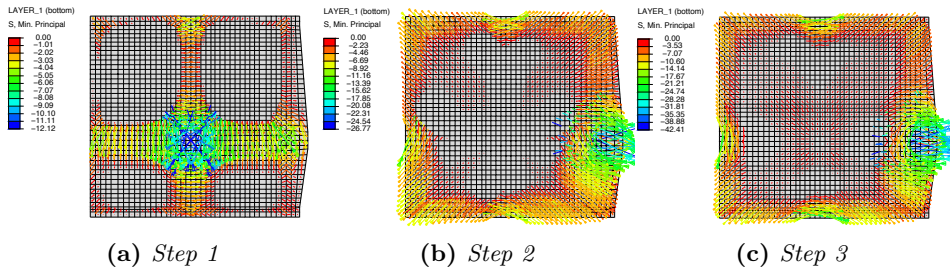


Figure 4.84: Compressive stresses in the bottom side of the slab - asymmetric sub-frame (units in MPa)

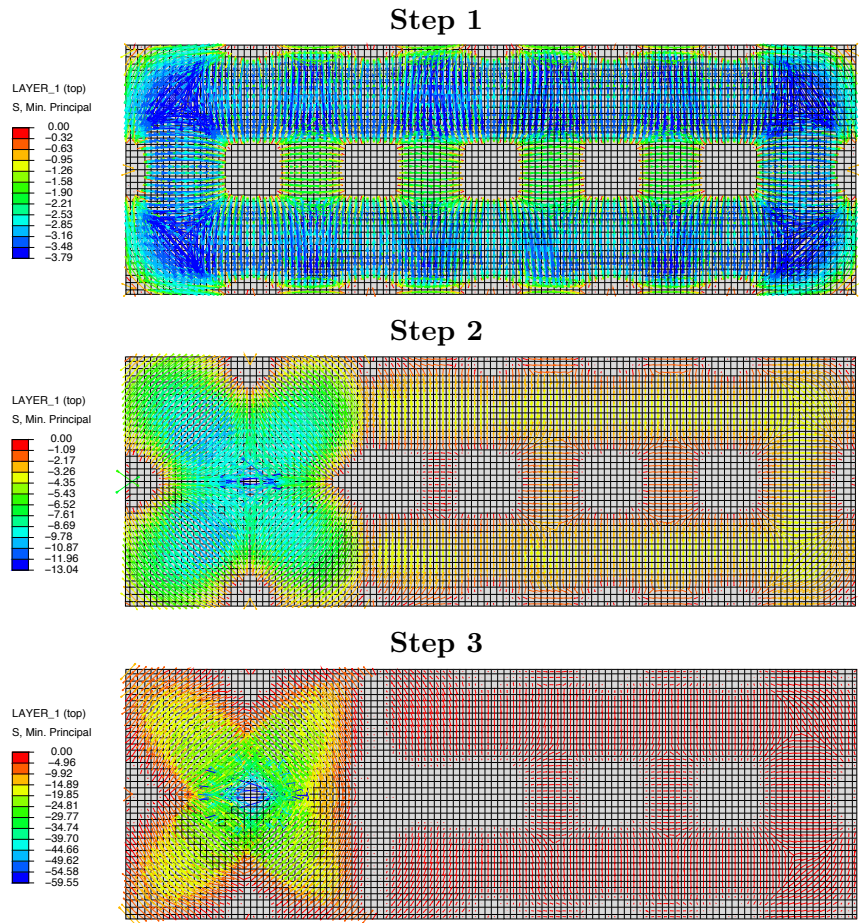


Figure 4.85: Compressive stresses in the top side of the slab - symmetric full-frame (units in MPa)

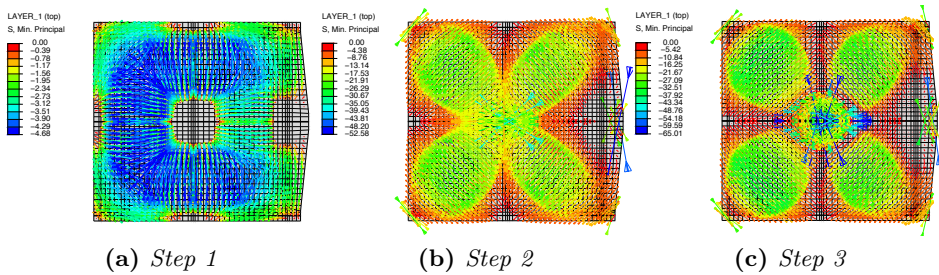


Figure 4.86: Compressive stresses in the top side of the slab - symmetric sub-frame (units in MPa)

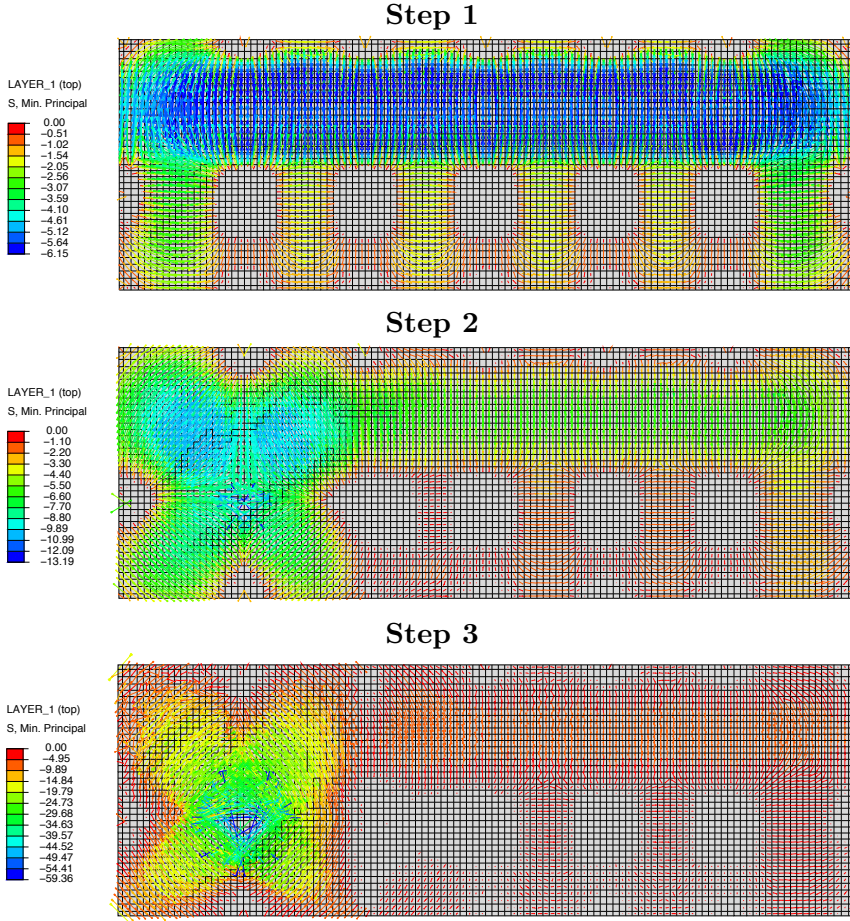
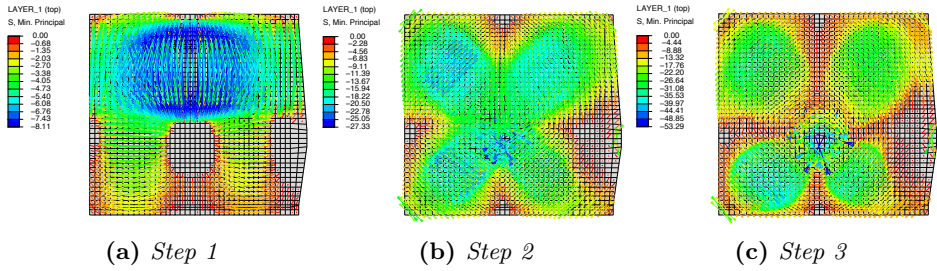


Figure 4.87: Compressive stresses in the top side of the slab - asymmetric full-frame (units in MPa)



4.6 Conclusions on the finite element models

The FE modelling using Abaqus/CAE environment allowed simulating the column removal scenario in steel and concrete composite structures, including material non-linearities which influence significantly the structural response. The numerical models were developed on the sub-frames and full-frames object of the studies of the 'RobustImpact' research project. The comparison between the experimental and numerical results for the main parameters enabled calibration of the models. The models of the sub-frames extend and complete the information gained from the experimental programme, and deliver more data related to local strains and stresses, internal forces and other quantities that could not be measured during the tests. In this work, the most important parameters to evaluate the structural response are reported. Furthermore, the validated FE models of the sub-frames are used not only to extend the data obtained from the tests, but also to gather results for studying different column removal scenarios and for developing numerical models of the entire full-frame structures. The models are built with 1D beam elements for the steel members and with 2D shell elements for the concrete slab, including two different options of beam-to-column connections: ideal hinges at the beam ends connected to the columns and semi-rigid connections accounting for the mechanical component of the joints as in the component method proposes by the EN1993-1-8 [2005]. Another factor of significant importance is the concrete modelling, both in compression and in tension; the frame response is strongly affected by the concrete slab. Comparing the numerical and experimental outcomes, it was demonstrated that the models with semi-rigid connections are more accurate than the models with pin connections, even if the pin joint models appeared to be adequate for studying the overall structural response of the frame. The results of the sub-frames models are compared to the experimental ones in terms of:

- load-deflection relationship of the removed column that is assumed as the parameter which characterizes the overall response of the structure;
- axial forces in the columns that allow understanding the load redistribution from the damaged to the undamaged part of the structure;
- beam-to-column rotations that allow evaluating the ductility and rotational demand of the joints;
- tensile damage patterns and stress distribution in the concrete slab that characterize the development of alternate load paths and force redistribution by the activation of membrane action.

The FE models approximated with good accuracy all quantities, even if some discrepancies were observed in the joint rotations and axial force distribution in the columns. Such differences are not significant considering the complexity of the frame behaviour, in particular of the joints and of the slab.

The FE modelling of the full-frame structures was performed based on the calibrated models of the sub-frames. The comparison between the results of the full-frames and sub-frames showed how the mechanisms developed in the frames after the column removal and the load redistribution are similar, even if studying substructures is more conservative than studying the overall building. The cracked regions in the slab in the sub-frames are more extended than in the full-frame because of the loss of force distribution among the storeys. Furthermore, the joint rotations in the sub-frames are smaller than those in the full-frame case.

The comparison of the results between the two structural configurations (*symmetric* and *asymmetric*) allowed concluding that the structural response is similar, with a stiffer response shown by the *asymmetric* frame.

Chapter 5

Influence of the location of the removed column on structural behaviour

5.1 Introduction

To date, most of the experimental studies on the effects of column removal on the response of building structures focused on the internal, or central, column removal scenario. Due to high experimental cost, there are few studies on beam-slab structures that investigate all the possible scenarios of column loss, for instance edge column and corner column. The studies that are available in literature comment that the corner column loss is the most critical scenario for progressive collapse due to a lack of lateral restraints from two sides of the frame (Tuan et al. [2017]). Additionally, the collapse of the penultimate column, which is next to the corner column, is considered as the second most critical scenario for progressive collapse investigations. The penultimate column loss leads to large inward movements of the adjacent columns (Dat and Tan [2013], Dat and Tan [2014a]), which in turn may cause progressive collapse.

This chapter reports on the results of numerical analyses on several scenarios of column removal, by means of the software Abaqus/CAE using the validated model described in Chapter 4. Six different ground floor column removal scenarios are considered in the analyses of the reference framed structures described in §4.2.1, including cases of loss of internal columns, edge columns and corner columns. In particular, the collapse of two internal columns (columns B2 and B3), three edge columns (columns C2, A2 and B1) and one corner column (column C1) was simulated, as illustrated in Figure 5.1. In each case, the directly affected area of the floor, shown in Figure 5.3, is different in terms of geometry and structural configuration. For more realistic modelling, a uniformly distributed load, equal to the de-

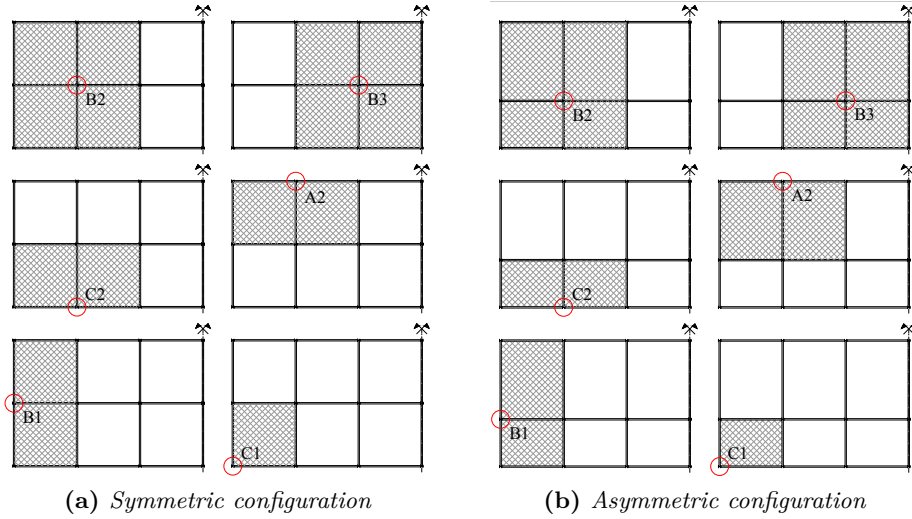


Figure 5.1: Column removal scenarios considered in the reference frames

sign load for the Ultimate Limit State (Baldassino et al. [2018]), was applied onto the slab before the column removal. For this purpose, in the analyses three loading steps are implemented as described in §4.2.2 and include the loading onto the slab, the column removal, and the application of a tensile force on the removed column to evaluate the residual strength of the system and to allow developing membrane forces in the slab. The analyses are stopped when a deflection of about 300 mm is achieved at the removed column (see Chapter 4). The results and the comparison between the cases focus on the overall response of the frames, on the columns, joints and slab system. Furthermore, the results on the full-frame structures are compared to the ones obtained from the analysis on the sub-frame systems for each column removal scenario. This comparison is helpful to understand if modelling substructures, more simple and computationally less complex, allows to evaluate the overall structural behaviour, force transmission and load redistribution within the frame.

5.2 Results of the analysis

5.2.1 The overall response

The graphs in Figure 5.3 show the comparison of the overall response of the *symmetric* and *asymmetric* full-frame structures. Figure 5.4 is related to the sub-frame cases. In particular, the axial force P on the removed column is expressed with respect to the force P_{max} at the end of the loading phase (*step 1*) as reported in Table 5.1.

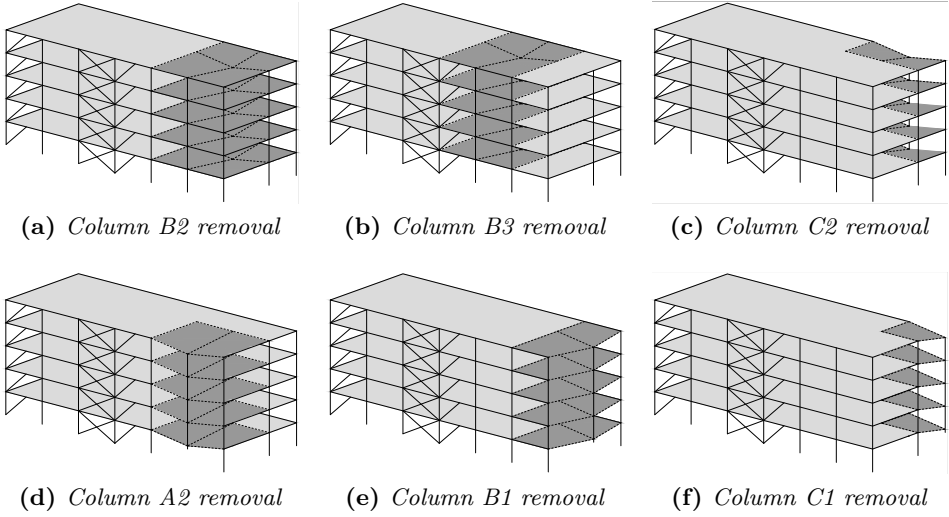


Figure 5.2: Representation of the affected part of the structure after column loss - asymmetric frame

Table 5.1: Axial force in the removed column at the end of the loading phase (step 1)

		P_{max} (kN)					
		B2	B3	C1	A2	C2	B1
Full-frame	Symm.	2476.8	2379.5	509.4	1043.6	1061.6	1197.0
	Asymm.	2545.7	2462.8	330.7	1346.6	710.5	1237.9
Sub-frame	Symm.	711.1	-	92.3	230.2	235.0	236.9
	Asymm.	743.0	-	44.0	316.0	139.1	252.8

As a first appraisal of the graphs, the corner column C1 removal case is the one with the greatest structural capacity: at a deflection of about 300 mm, the force at the removed column is about 1.8 and 3.8 greater than the axial force acting at the end of *step 1* for the *symmetric* and *asymmetric* full-frame, respectively (Figure 5.3). The *asymmetric* sub-frame presents similar results, while the *symmetric* one exhibits a smaller structural capacity compared to the full-frame case (Figure 5.4). The internal column B2 case is the one with the smallest structural capacity among the six considered scenarios, similar to the column B3 case. In the *asymmetric* sub-frame (Figure 5.4 (b)), the case with the least capacity is the edge column A2. The symmetry of the frame confirms the similar behaviour between the edge columns removal scenarios (A2, C2 and B1); on the other hand, the asymmetric effects enable a stiffer response of the edge column C2 compared to the opposite edge column A2.

A more clear understanding on the structural capacity is given in Table 5.2,

Table 5.2: Variation in % of the ratio P/P_{max} for the column removal cases with respect to the corner column C1 removal case at the end of the (step 3)

		P/P_{max}		Variation (%)	
Case		Symm.	Asymm.	Symm.	Asymm.
Full-frame	Corner C1 removal	-1.79	-3.76		
	Internal B2 removal	-0.73	-0.82	-145	-359
	Internal B3 removal	-0.97	-1.08	-85	-248
	Edge A2 removal	-1.48	-1.00	-21	-276
	Edge C2 removal	-1.51	-3.34	-19	-13
	edge B1 removal	-1.24	-1.35	-44	-179
Sub-frame	Corner C1 removal	-1.07	-4.17		
	Internal B2 removal	-0.57	-0.72	-88	-479
	Edge A2 removal	-0.67	-0.36	-60	-1058
	Edge C2 removal	-0.60	-2.64	-78	-58
	Edge B1 removal	-0.62	-0.83	-73	-402

where the ratio P/P_{max} at the end of *step 3* and the variation in percentage of this ratio for each column removal case with respect to the corner column C1 removal case are reported. The ratio P/P_{max} in the corner column C1 removal case is 145 % and 359 % greater than the internal column B2 case in the *symmetric* and *asymmetric* full-frame, respectively. The variation is 85 % (*symmetric*) and 248 % (*asymmetric*) in the internal column B3 removal case. Considering the edge column removal cases, the variation with respect to the corner column C1 case is smaller than the internal column cases.

The results in Figure 5.3 and Figure 5.4 show that the lack of lateral restraints in the edge or corner columns removal case does not significantly influence the structural response. The absence of restraints does not allow the development of catenary action in the beams and full membrane forces in the slab, but the contribution of the grillage system is crucial (Stylianidis et al. [2016a]). Furthermore, it is important to notice that edge and corner columns sustain smaller loads than internal columns (Table 5.1), thus contributing to increase structural capacity after column loss.

The results in these graphs show that the corner column removal scenario is not the most critical one, presenting the greatest structural capacity compared to the other cases. Full compressive and tensile membrane actions could not be mobilised, but the edge beams ensure a significant contribution to the overall response and contributes to the formation of limited tensile membrane action in the slab. Furthermore, hogging bending moment regions develop along the diagonals connecting edge columns B1 and C2 as in the study of Tuan et al. [2017], as confirmed by the analysis of the tensile damage pattern in the slab (Figure 5.5).

The comparison between full-frame and sub-frame analyses shows that the full building exhibits a stiffer response than the substructure. The sub-frames eliminate the effects of framing action from the members of the sto-

ries above. Column loss in the lower floors of high rise building mobilize the structure above, efficiently redistributing the gravity loads and contributing to enhance the structural capacity. This aspect suggests that buildings with a small number of floors or high rise buildings in which the collapse of a column occurs in the upper stories are more vulnerable to progressive collapse than high rise buildings affected by a column loss in the lower stories, confirming the studies of Li and El-Tawil [2013a].

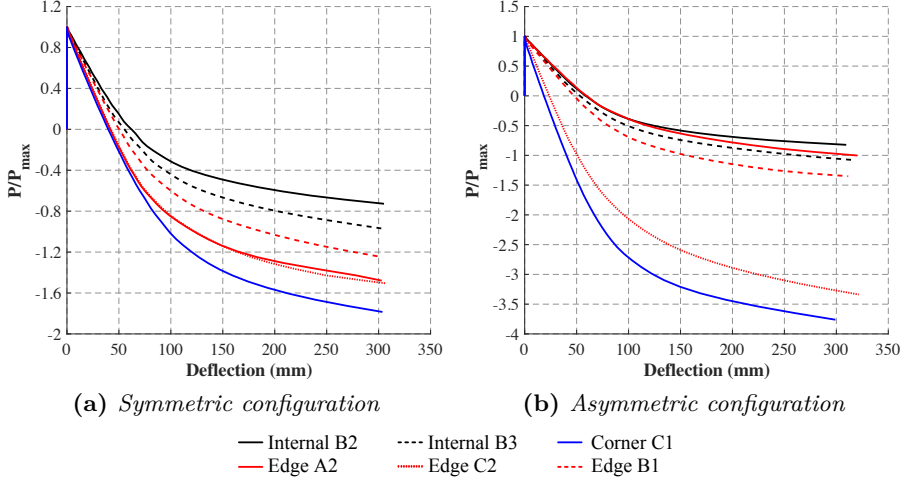


Figure 5.3: Structural responses from different column loss scenarios - full-frame structures

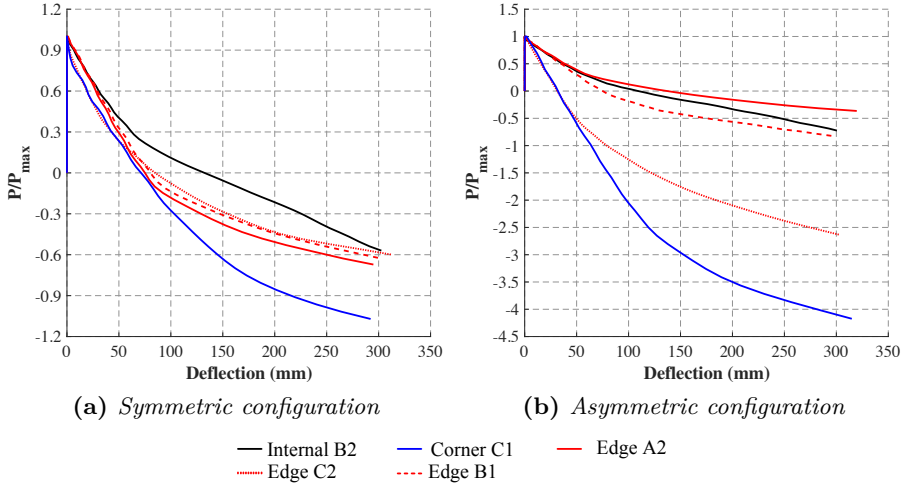


Figure 5.4: Structural responses from different column loss scenarios - sub-frame structures

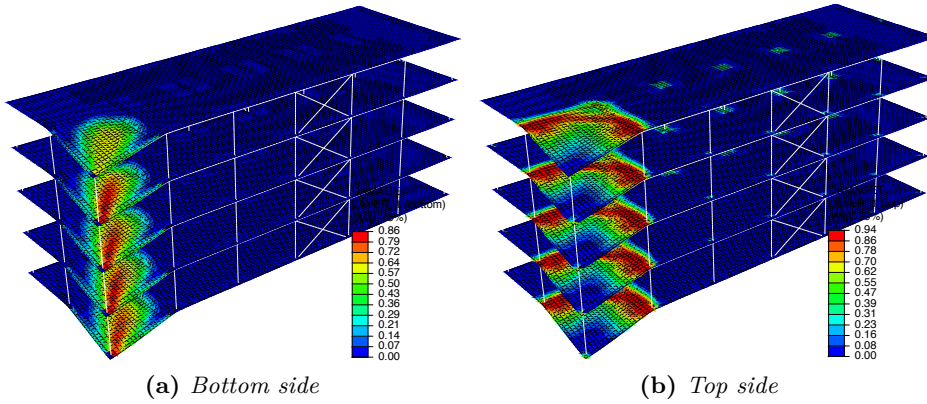


Figure 5.5: Tensile damage pattern at the end of the step 3 - symmetric full-frame

5.2.2 Loads redistribution and the role of the concrete slab

The role of the concrete slab and of the frame continuity is important for the gravity loads redistribution from the damaged to the undamaged part of the structure after the column loss, as confirmed by the analyses of the axial forces in the columns and of the stresses distribution in the slab. Considering the removal of column B2, the loads are redistributed along the direction shown in Figure 5.6, increasing the axial forces in the columns as reported in Figure 5.7 and Figure 5.8 for the full-frames and sub-frames, respectively. The results relative to the other column removal cases are reported in the Appendix A. The variation of the force in the columns positioned close to the removed column is relevant, both at the end of *step 2* and *step 3* (Figure 5.7); while the variation in frames 4 to 7 is limited. Therefore, the columns surrounding the affected zone of the structure (Figure 5.2) are directly influenced by the event, sustaining a force at the end of *step 3* that is significantly greater than the force acting before the column removal. The force increase is up to about 360 % and 666 % at the base of column C1 in the *asymmetric* full-frame and sub-frame structures, respectively, as shown in Figure A.16 and Figure A.17 for the column B1 loss scenario. The force increase in the same column in the *symmetric* frames is more modest than the *asymmetric* case. The other columns are minimally affected by the event, and the variation is positive (increase) or negative (decrease) depending by their position in the frame.

In the edge columns loss scenarios (Figure A.6, Figure A.11 and Figure A.16), the loads are redistributed along the direction in which catenary action can be developed in the beams (e.g. from column C2 to column C1 and C3). The load redistribution in the transverse direction is small due to the absence of lateral restraints along the perimeter of the frame. In the case of loss of a corner column, the direction along the strong axis of the columns

is the one with the higher force transmission (from column C1 to C2 in Figure A.21). The comparison between the *symmetric* and *asymmetric* frame (e.g. in Figure A.2) shows a quite different loads redistribution mainly due to two aspects: i) the different loading area acting per each column; ii) the higher force transmission among the shorter, and stiffer, frame span than the longer ones.

The stresses in the slab (Figure 5.9 and Figure 5.10) clearly indicated the mechanism of force redistribution in the structure. In the bottom side of the slab, cracks develop along the lines connecting the removed column with the surrounding columns in the diagonal directions (e.g. Figure 5.11), confirming the theory of the yield lines (Jones and Wood [1967]). In these zones, the tensile stresses are null due to the presence of the fully cracked region (e.g. Figure A.3). In the top side of the slab, full or partial compressive concentric rings develop, as illustrated in Figure A.3 and Figure A.8. Along the perimeter of the affected zone, the slab is in tension and in compression in the bottom and top side, respectively, indicating the development of hogging bending moment (Figure 5.12). In these regions, the structural resistance is primarily governed by the flexural capacity of the slab and of the beams, and by the torsional resistance of the perimeter beams. The reinforcing bars are called to sustain important tensile forces and, close to the columns, additional rebars are needed to ensure an efficient force transmission from the slab to the columns, as shown in §4.3.1.

The cracking pattern and the stress distribution in the slab obtained from the column removal analyses are similar to those obtained from the studies of Lim et al. [2017], Lu et al. [2017a] and Yu et al. [2018a]. The presence of the slab allows developing new load paths re-balancing gravity loads and mitigating the effect of column loss. These observations indicate the importance of the concrete slab to achieve a robust structural response and enhance the progressive collapse resistance of structures.

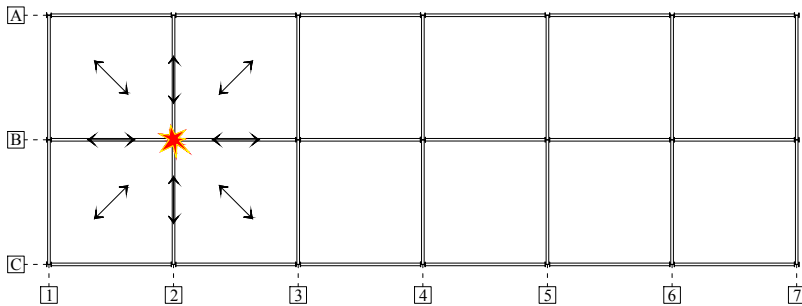


Figure 5.6: Direction of the load redistribution - column B2 removal

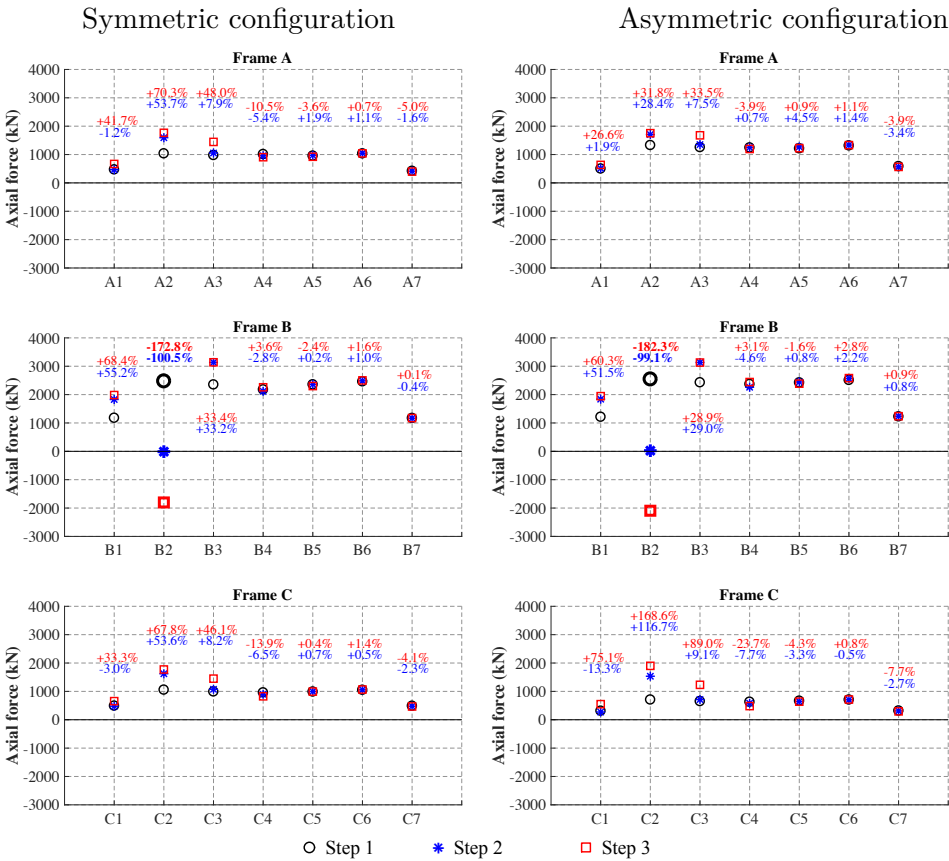


Figure 5.7: Axial force in the columns - full-frame - floor 1 - column B2 removal

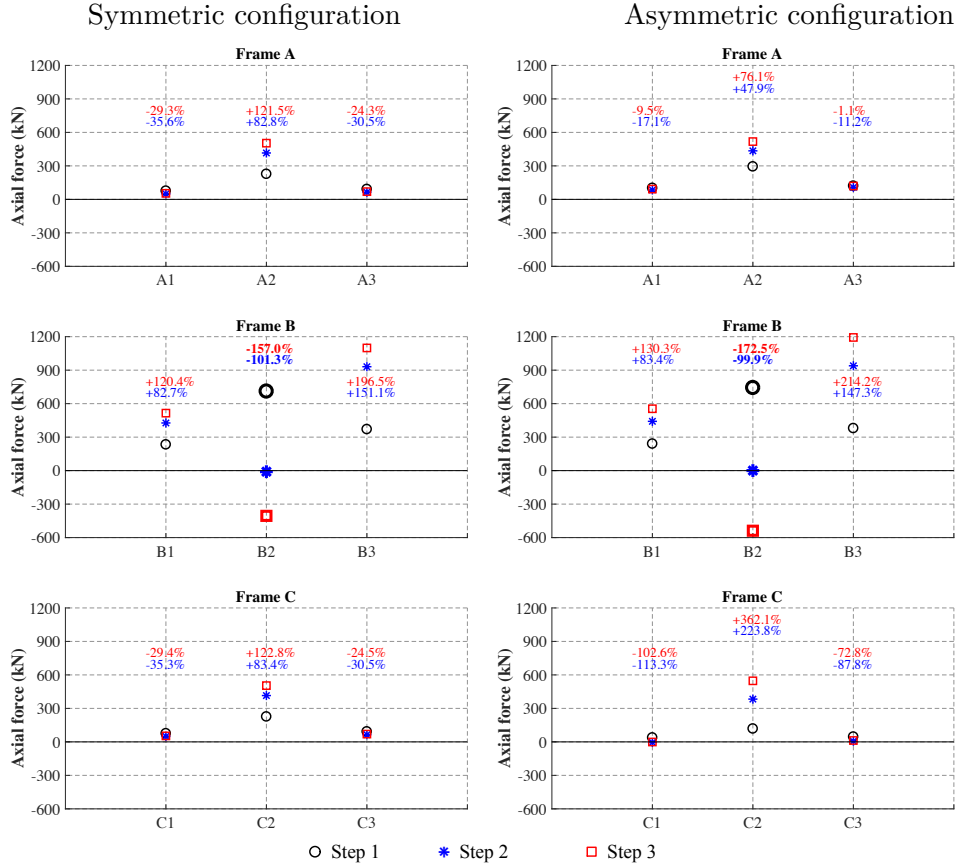
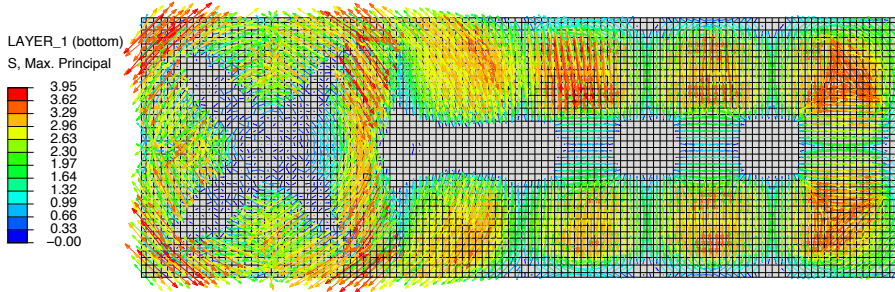
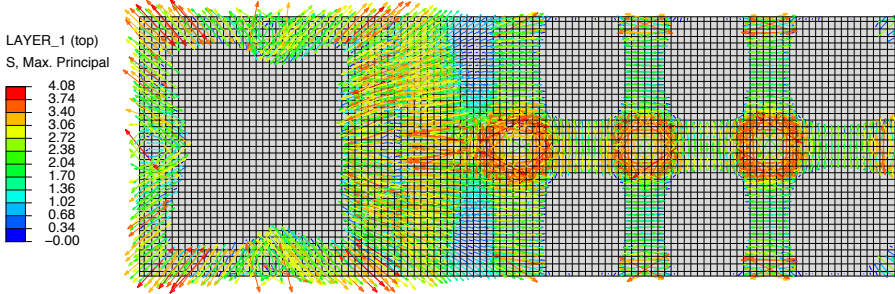


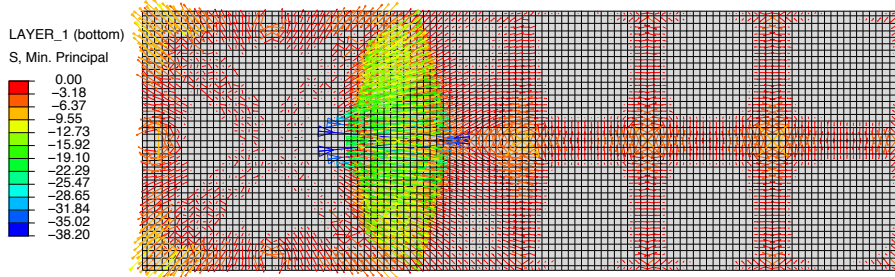
Figure 5.8: Axial force in the columns - sub-frame - column B2 removal



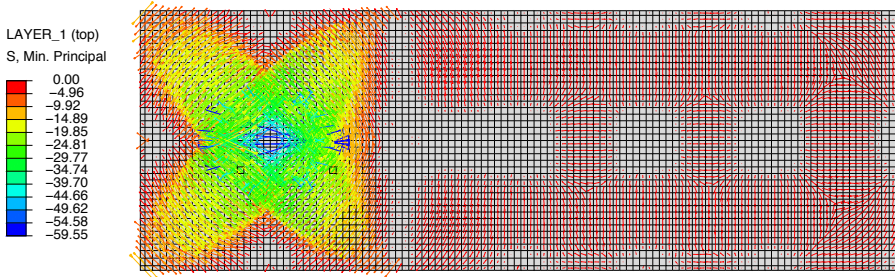
(a) Tensile stresses - bottom side



(b) Tensile stresses - top side

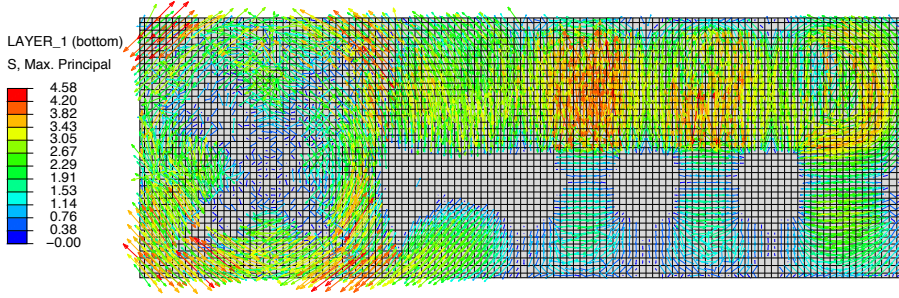


(c) Compressive stresses - bottom side

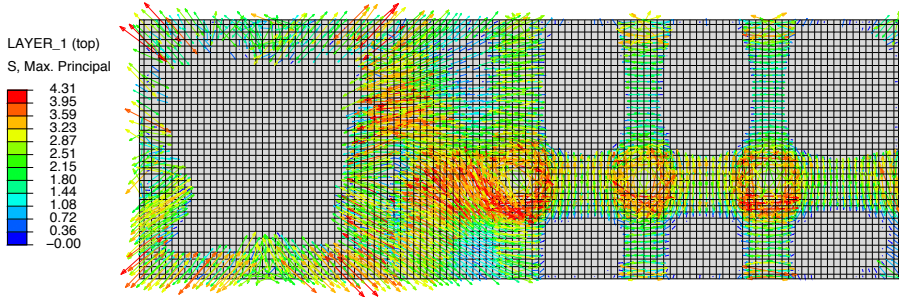


(d) Compressive stresses - top side

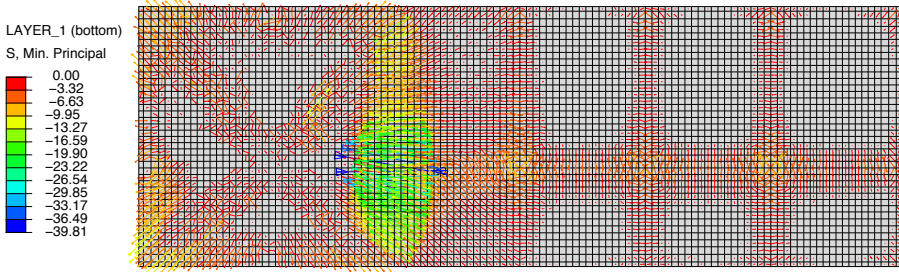
Figure 5.9: Stresses in the slab - symmetric configuration - column B2 removal (units in MPa)



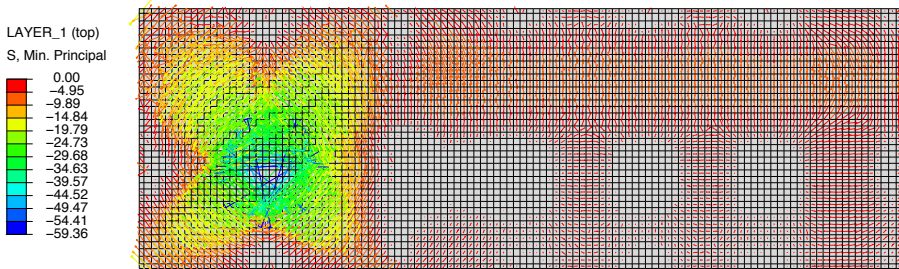
(a) *Tensile stresses - bottom side*



(b) *Tensile stresses - top side*



(c) *Compressive stresses - bottom side*



(d) *Compressive stresses - top side*

Figure 5.10: Stresses in the slab - asymmetric configuration - column B2 removal (units in MPa)

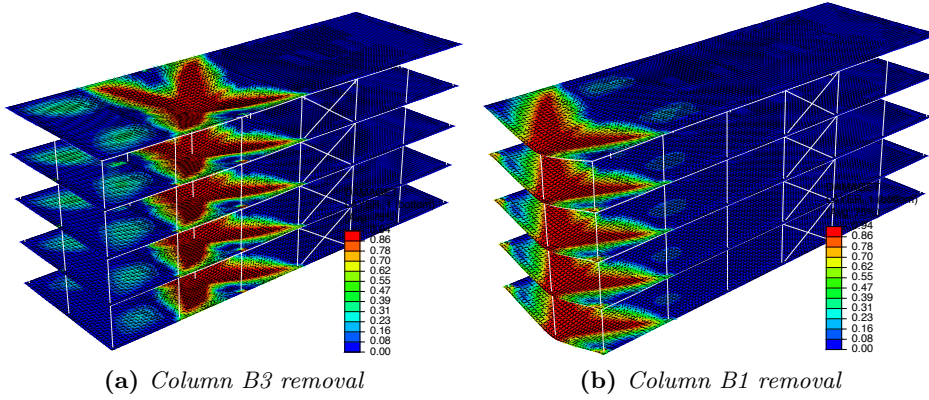


Figure 5.11: Tensile damage pattern in the bottom side of the slab - symmetric full-frame

5.2.3 Joints rotation

In this section, the joint rotations obtained from the analyses of the different column removal scenarios are reported and commented. The results allow quantifying the rotation demand at the end of the column removal and at a deflection of about 300 mm of the removed column.

Each joint is named as follows: the first two terms identify the beam, while the third term identifies the end of the beam where the joint is positioned (e.g. B1-B2_B1 identifies the joint that connects the beam B1-B2 to the column B1). The values reported for each joint comprise of the three steps of the analysis as described in the previous sections. The nomenclature of the structural elements is illustrated in Figure 4.1 and Figure 4.24 for the full-frames and sub-frames, respectively, while Figure 4.35 defines the positive sign of the rotation.

The experimental data reported in the Chapter 4 clearly indicated that a rotation of the beam-to-column connections of about 70 mrad can be assumed as the rotation for which the rupture of the bolts of the bottom row take place. In this context, a measure of the required ductility of the joint can be assessed.

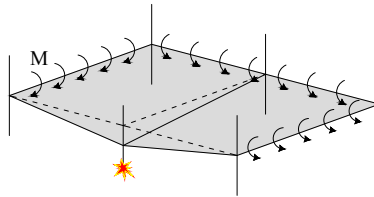


Figure 5.12: Hogging bending moment along the perimeter of the affected zone

The results reported in the following tables show that the rotation in all the joints at the end of *step 1* and *step 2* are relatively small compared to the rotation at the end of *step 3*. In most cases, the rotations do not overcome the value of 70 mrad, even if the column B1 removal case in the *asymmetric* sub-frame configuration cause important rotations in some joints, as shown in Table 5.7. In particular, at the end of *step 3* the rotations are approximately of 80 mrad in all the joints, and of approximately 110 mrad in the joint B1-C1_C1.

These results allow concluding that the column collapse provide similar joints rotations among the scenarios considered in this study, even if the column B1 removal case is more critical with respect to the other cases.

Table 5.3: Joints rotations - column B2 removal (units in mrad)

Joint		Symm. configuration		Asymm. configuration	
		Full-frame	Sub-frame	Full-frame	Sub-frame
B1-B2_B1	Step 1	3.2	3.9	3.3	4.1
	Step 2	13.8	26.0	12.8	21.3
	Step 3	58.6	49.6	40.5	54.9
B1-B2_B2	Step 1	-1.4	-2.9	-1.5	-3.5
	Step 2	1.3	15.0	1.2	6.2
	Step 3	23.2	45.6	13.5	43.9
B2-B3_B2	Step 1	-0.8	-2.3	-0.8	-3.2
	Step 2	2.3	16.2	2.0	8.7
	Step 3	32.7	46.4	17.8	51.5
B2-B3_B3	Step 1	1.0	2.8	1.0	3.0
	Step 2	9.6	22.8	8.6	17.3
	Step 3	41.3	47.3	27.5	47.4
A2-B2_A2	Step 1	3.6	4.2	7.0	8.6
	Step 2	15.1	28.6	16.8	24.3
	Step 3	59.2	53.5	40.3	51.7
A2-B2_B2	Step 1	-1.2	-2.0	-5.1	-7.7
	Step 2	1.6	12.9	-4.4	-6.1
	Step 3	26.4	43.0	4.7	13.5
B2-C2_B2	Step 1	-1.2	-2.0	1.6	-0.5
	Step 2	1.6	13.3	8.7	16.5
	Step 3	26.1	45.4	35.7	56.7
B2-C2_C2	Step 1	3.4	4.2	1.1	0.7
	Step 2	15.0	28.6	14.3	22.1
	Step 3	59.3	53.2	50.1	67.1

5.3 Concluding remarks

The influence of the location of the removed column on the structural response, which was the focus of this chapter, allows to draw the following conclusions:

Table 5.4: Joints rotations - column B3 removal (units in mrad)

Joint		Symm. configuration		Asymm. configuration	
		Full-frame	Sub-frame	Full-frame	Sub-frame
B2-B3_B2	Step 1	0.3	-	0.3	-
	Step 2	6.5	-	6.3	-
	Step 3	33.2	-	34.4	-
B2-B3_B3	Step 1	-0.6	-	-0.6	-
	Step 2	0.3	-	0.3	-
	Step 3	16.4	-	17.7	-
B3-B4_B3	Step 1	-0.6	-	-0.6	-
	Step 2	1.3	-	1.2	-
	Step 3	21.5	-	22.2	-
B3-B4_B4	Step 1	0.6	-	0.6	-
	Step 2	6.6	-	6.7	-
	Step 3	35.5	-	36.4	-
A3-B3_A3	Step 1	3.3	-	6.5	-
	Step 2	12.4	-	14.7	-
	Step 3	55.3	-	52.7	-
A3-B3_B3	Step 1	-0.7	-	3.7	-
	Step 2	0.6	-	6.1	-
	Step 3	16.7	-	0.5	-
B3-C3_B3	Step 1	-0.6	-	2.1	-
	Step 2	0.7	-	7.3	-
	Step 3	15.2	-	38.9	-
B3-C3_C3	Step 1	3.0	-	0.9	-
	Step 2	12.0	-	12.2	-
	Step 3	55.0	-	73.0	-

- the corner column loss case is the one with the greatest structural capacity compared to the other cases: the ratio P/P_{max} between the force acting in the column and the maximum reaction force at the end of the loading phase (*step 1*) is 145 % (*symmetric* full-frame) and 359 % (*asymmetric* full-frame) higher than the values obtained for the internal column B2 removal case;
- the *asymmetric* frame subjected to the corner column C1 removal and the edge column C2 removal exhibits a stiffer response compared to the *symmetric* frame;
- the edge column removal cases in the *symmetric* structure present similar results between them, confirming the symmetry of the frame;
- the structural capacity of the sub-frames is smaller than the capacity of the full-frame structures due to the absence of the upper stories and of the frame continuity;
- the joint rotations at the end of the column removal are significantly smaller than the rotation at the end of the analyses (*step 3*). Most of

Table 5.5: Joints rotations - column C2 removal (units in mrad)

Joint		Symm. configuration		Asymm. configuration	
		Full-frame	Sub-frame	Full-frame	Sub-frame
C1-C2_C1	Step 1	1.6	1.2	1.4	1.5
	Step 2	7.0	12.4	4.3	7.4
	Step 3	29.6	56.3	31.2	53.4
C1-C2_C2	Step 1	-0.5	-1.4	-0.5	-0.8
	Step 2	0.2	3.5	0.0	1.3
	Step 3	16.7	49.1	18.8	49.2
C2-C3_C2	Step 1	-0.1	-1.2	0.0	-0.6
	Step 2	1.2	4.9	0.7	1.8
	Step 3	19.1	53.3	21.1	53.7
C2-C3_C3	Step 1	0.3	0.8	0.2	1.1
	Step 2	4.4	10.4	2.4	6.7
	Step 3	31.9	50.4	32.8	48.9
B2-C2_B2	Step 1	0.8	1.7	-1.9	-0.1
	Step 2	4.2	9.1	1.0	6.2
	Step 3	27.5	49.0	34.8	51.4
B2-C2_C2	Step 1	-3.4	-5.9	-1.2	-1.5
	Step 2	3.6	6.7	3.9	6.8
	Step 3	40.1	54.2	57.3	75.1

Table 5.6: Joints rotations - column A2 removal (units in mrad)

Joint		Symm. configuration		Asymm. configuration	
		Full-frame	Sub-frame	Full-frame	Sub-frame
A1-A2_A1	Step 1	1.5	2.3	1.7	1.4
	Step 2	6.9	19.1	9.3	39.0
	Step 3	32.3	58.4	33.4	59.0
A1-A2_A2	Step 1	-0.5	-1.4	-0.6	-1.4
	Step 2	0.2	1.5	0.5	27.3
	Step 3	19.4	40.9	16.5	49.2
A2-A3_A2	Step 1	-0.1	-0.6	-0.1	-1.3
	Step 2	1.2	4.0	1.6	30.6
	Step 3	22.8	48.2	22.9	52.7
A2-A3_A3	Step 1	0.2	1.7	0.2	1.3
	Step 2	4.3	15.6	6.4	34.9
	Step 3	36.3	49.7	36.6	52.2
A2-B2_A2	Step 1	-3.6	-4.7	-6.9	-10.0
	Step 2	3.3	10.8	1.1	20.6
	Step 3	47.6	51.2	33.5	36.7
A2-B2_B2	Step 1	0.6	1.5	3.5	6.9
	Step 2	4.1	9.7	7.1	27.7
	Step 3	31.9	41.4	26.4	39.4

the joints do not achieve the 'critical' rotation of 70 mrad, even if the column B1 removal case in the *asymmetric* sub-frame provides higher rotations compared to the other cases.

Table 5.7: Joints rotations - column B1 removal (units in mrad)

Joint		Symm. configuration		Asymm. configuration	
		Full-frame	Sub-frame	Full-frame	Sub-frame
A1-B1_A1	Step 1	1.9	2.0	4.1	4.9
	Step 2	10.7	16.4	11.4	24.4
	Step 3	44.7	44.5	40.5	78.8
A1-B1_B1	Step 1	-0.4	-0.6	-2.2	-3.8
	Step 2	0.8	4.8	-4.7	-0.7
	Step 3	18.0	34.4	3.2	20.4
B1-C1_B1	Step 1	-0.4	-0.6	1.3	0.4
	Step 2	0.7	4.7	6.0	19.8
	Step 3	17.7	34.7	38.1	81.4
B1-C1_C1	Step 1	1.6	2.0	0.2	-0.2
	Step 2	10.4	16.9	10.4	28.9
	Step 3	44.6	45.2	64.2	112.4
B1-B2_B1	Step 1	-3.1	-4.4	-3.1	-5.1
	Step 2	3.6	8.5	3.0	17.0
	Step 3	31.6	37.6	37.6	80.6
B1-B2_B2	Step 1	0.9	2.7	0.9	3.0
	Step 2	5.6	12.6	5.2	18.2
	Step 3	31.3	38.6	30.8	78.5

Table 5.8: Joints rotations - column C1 removal (units in mrad)

Joint		Symm. configuration		Asymm. configuration	
		Full-frame	Sub-frame	Full-frame	Sub-frame
B1-C1_B1	Step 1	0.3	-0.4	-1.4	-0.7
	Step 2	4.1	6.9	1.4	8.4
	Step 3	31.2	47.5	44.0	54.8
B1-C1_B1	Step 1	-2.1	-4.7	-0.6	-0.2
	Step 2	4.2	8.5	3.5	10.4
	Step 3	48.9	58.4	67.4	76.4
C1-C2_C1	Step 1	-1.8	-4.8	-1.5	-1.9
	Step 2	2.9	8.2	0.7	6.9
	Step 3	37.9	53.1	41.4	56.8
C1-C2_C2	Step 1	0.4	0.1	0.4	1.0
	Step 2	3.9	8.1	2.0	6.0
	Step 3	33.2	51.7	30.7	48.4

The columns close to the affected part of the structure are directly influenced by the event, supporting a load significantly greater than the load acting before the column removal; the axial force variation in the other columns is limited. The stress distribution in the concrete slab provide a clear understanding of the force redistribution in the structure, and allow showing that membrane forces develop, enhancing the structural capacity of the system and achieving a progressive collapse resistance.

Chapter 6

Development of membrane action in concrete slabs

6.1 Introduction

This chapter focus on the behaviour of flat concrete slabs subjected to large displacement caused by a column loss scenario in framed structures. In the first part of the chapter, the structural mechanisms developed in the slabs at large deflection are described and the main analytical approaches and solutions available in literature to evaluate the static response of reinforced concrete slabs, including the effects of membrane action, are presented. Furthermore, in the second part, a new analytical method, based on two models developed by Omer [2006] and Herraiz Gómez [2016], to determine the load-deflection relationship of laterally restrained simply supported square reinforced concrete slabs is introduced.

6.2 Behaviour of slabs subjected to large displacements

Column collapse in framed structures causes large deformations of the flooring system and large rotations in the beam-to-column connections. A robust structural response of the frame can be achieved if the structural members have sufficient resistance and ductility in order to redistribute forces from the damaged to the undamaged parts of the structure, allowing a new stable equilibrium configuration. In steel-concrete composite structures and in reinforced concrete structures, the contribution of the floor system to the overall structural resistance is important. In progressive collapse studies, neglecting the slab behaviour can lead to an uneconomical design of beam and column sections (Pham et al. [2017a]). Moreover, catenary action in beams and tensile membrane action in slabs are two efficient mechanisms

to enhance structural capacity at large deformation stage. Catenary action in beams is usually associated with the change of the axial force in the beams from compression to tension and with combined bending moment and axial force in the connections with the columns. The load-deflection response of the beam from the initial elastic phase up to its defined failure can be obtained by combining the beam analyses with the joint behaviour (Stylianidis et al. [2016b]). The full response of axially restrained beams comprises several stages as shown in Figure 6.1. After the initial elastic phase, the development of some inelastic regions causes a softening. During this stage, noted as compressive arching stage, axial forces start to develop in the beam. After that, the axial force switches from compressive to tensile through a transient tensile stage, followed by the full tensile catenary stage if the connections have sufficient ductility.

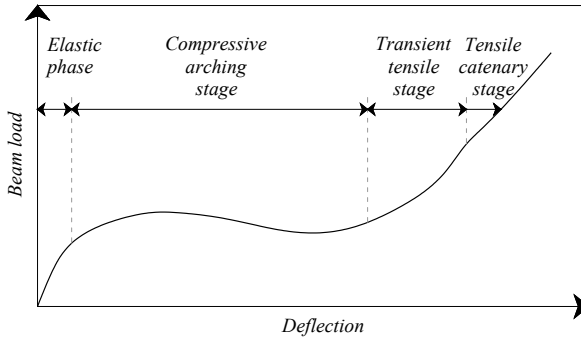


Figure 6.1: Static load-deflection response of axially restrained beams

Although 2D framed structures subjected to large displacement due to internal column removal are relatively simple to study because of the development of tensile catenary action in the beams, 3D framed buildings with flat slabs exhibit a complex structural behaviour after a column loss because of the development of self-balanced membrane forces (Herraiz Gómez [2016]). The development of tensile membrane action is indicated by the formation of a tensile zone in the middle of the slab and a peripheral compressive ring equilibrating the tensile net (Dat and Hai [2013]) as illustrated in Figure 6.2. The response of slabs subjected to peripheral (edge and corner) or internal column removal is different. Self-balanced membrane action can not be developed in the first ones, even if Pham et al. [2017a] demonstrated that, for the corner column loss case, the presence of peripheral beams helps to reduce the slab deformation in the edges, resulting in the development of a partial compressive ring and a limited tensile membrane zone in the central region of the slab.

To date, most of the methods to evaluate the failure of concrete slabs are based on the yield line theory developed by Johansen (Hognestad [1953]).

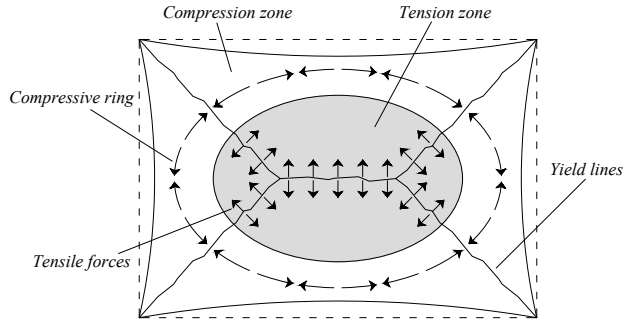


Figure 6.2: Development of membrane forces in laterally unrestrained slabs

This theory is a plastic theory in which the failure mechanism of the slab is firstly postulated, and the ultimate load can be obtained by means of the principle of virtual works resulting in an upper bound solution. According to this theory, the cracks and the curvature of the slab when the ultimate load is reached, are located along lines named as yield lines. The parts of the slab bounded by these lines remain elastic and planar, and rotate rigidly around the yield lines. The ultimate load predicted by this theory is lower than the load obtained experimentally due to neglecting the strain hardening of the reinforcement steel and the development of membrane forces in the slab.

The yield line pattern is affected by several parameters, such as the plastic moment capacities of the slab sections, the support conditions and the geometry of the slab. An illustration of the yield line pattern for different isotropically reinforced concrete slabs and boundary conditions is given in Figure 6.3. The load-deflection response of the slab depends on the lateral restraining system, and, in particular, on restraining planar movement along the edges as illustrated in Figure 6.4.

The yield line theory is assumed as the basis for developing methods to evaluate the resistance of the slab after the achievement of the ultimate load. Several authors proposed different analytical approaches to determine the static behaviour of restrained and unrestrained concrete slabs subjected to large displacements considering the development of membrane action. The main approaches are presented and described in the following section.

6.3 Overview of previous methods for the evaluation of the behaviour of restrained concrete slabs

During the last several decades, several authors developed different methods to evaluate the structural response of concrete slabs subjected to large displacements. Most of these methods focused on the response of slabs in

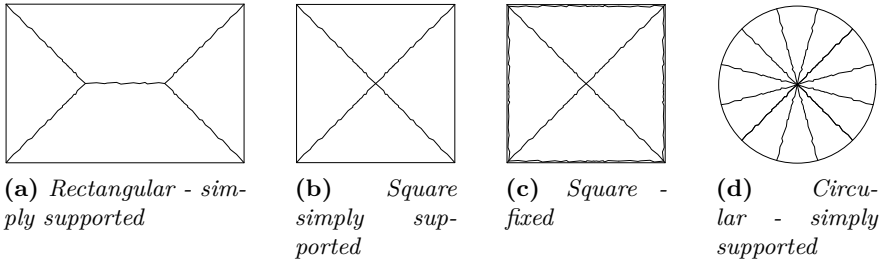


Figure 6.3: Example of yield lines for different support conditions and geometry

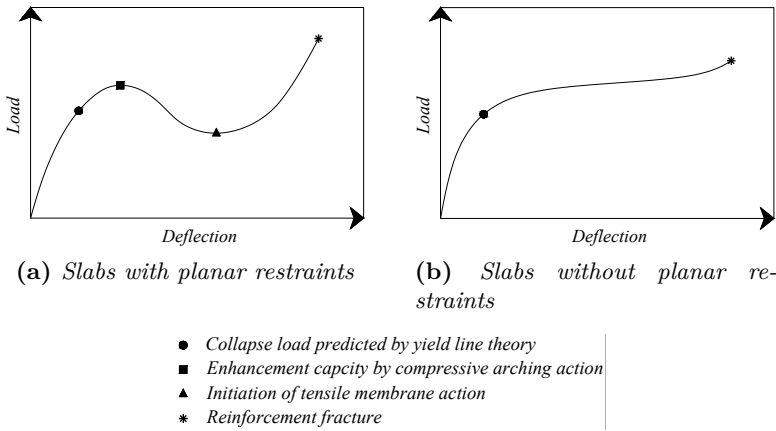


Figure 6.4: Load-deflection response of restrained and unrestrained concrete slabs (Omer [2006])

the membrane stage after the formation of the yield lines corresponding to the achievement of the ultimate load. The authors studied different loading conditions and boundary configurations such as square or rectangular slabs laterally restrained and unrestrained, and subjected to ambient or elevated temperature. Furthermore, one or two layers of reinforcement are taken into consideration. These methods allow evaluating the static load-deflection response of the slab, providing an helpful tool for the quantification of the residual strength after the formation of the collapse mechanism.

Morley [1967] extended the conventional yield line theory for reinforced concrete slabs allowing for membrane effects and moderately large deflections. For a slab with an assumed pattern of yield lines, the author developed a method to evaluate the relationship between load-carrying capacity and deflection of the slab. The yield line theory is modified by isolating the membrane displacement rates of the rigid portion of the slab that do not affect the yield line position, and developing a method choosing these rates so that the load estimated from the work-rate equation is stationary.

Park [1964a] developed a theory for the load carried at large deflections

by the tensile membrane action of rectangular concrete slabs subjected to a uniformly distributed load with edges restrained against all translations. The author assumed that the reinforcement acts as a plastic tensile membrane (Figure 6.5) ignoring the strain hardening of the steel and the failure of the slab. The load carried by the tensile membrane action can exceed the ultimate flexural strength of the slab. Furthermore, Park [1964b] investigated the effect of the compressive membrane action on reinforced concrete slabs. The edges of the slab are considered to be fixed against rotations and vertical translations, and a strip approximation is used to evaluate the load-deflection response of the slab as illustrated in Figure 6.6. Park [1964b] proposed an empirical value for the deflection at ultimate load based on experimental observations. The author assumed that the ultimate load of the slab corresponds to a deflection of about half of the slab thickness.

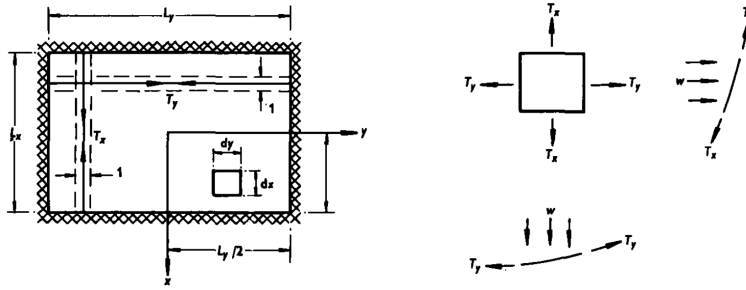


Figure 6.5: Details of plastic tensile membrane and forces acting on an infinitesimal slab element (Park [1964a])

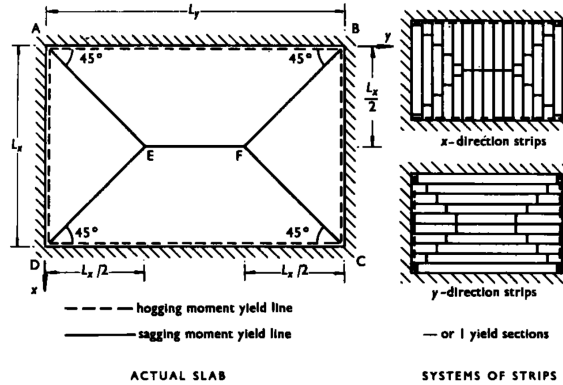


Figure 6.6: Assumed yield line pattern for slabs with restrained edges (Park [1964b])

Kemp [1967] proposed an upper bound solution for the yield load of a simply supported reinforced concrete square slab accounting for the effects of membrane forces. The method is valid for slabs subjected to uniformly

distributed load and reinforced in the bottom face only. The solution is based on an assumed rigid, perfectly-plastic slab mechanism. The formation of yield lines along the diagonals is consistent with the yield line theory, and the determination of the position of the neutral axis along the yield lines is based on geometrical and equilibrium considerations, hypothesizing the kinematic model as illustrated in Figure 6.7. The method proposed by Kemp allows determining the load-deflection response of the slab through equilibrium of an isolated slab segment ignoring the horizontal shear forces along the yield lines for the symmetry effects.

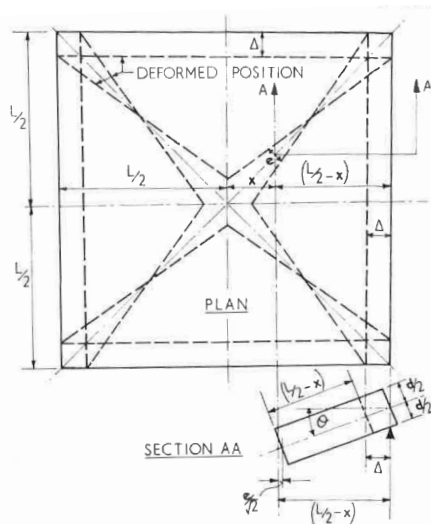


Figure 6.7: Deformations of the rigid plastic mechanism after Kemp [1967]

Sawczuk and Winnicki [1965] studied the plastic behaviour of simply supported reinforced concrete slabs at moderately large deflections. The behaviour of plastic concrete plate structures beyond the bending collapse load is influenced by the changes in geometry during the process of plastic deformation. The authors proposed an analytical approach based on a rigid perfectly-plastic slab mechanism to evaluate the behaviour of uniformly loaded simply supported rectangular slabs with one isotropic layer of reinforcement. The principle of virtual works is used to evaluate the load-deflection response without considering equilibrium equations. Sawczuk and Winnicki assumed two collapse mechanisms including the formation of a single axial crack perpendicular to the longer side of the slab (Figure 6.8 (a)) and the formation of two axial cracks perpendicular to the longer side at the intersection with the positive yield lines (Figure 6.8 (b)).

Janas [1968] studied the plastic response of reinforced concrete slabs, including membrane effects and geometry changes at large deflections. For

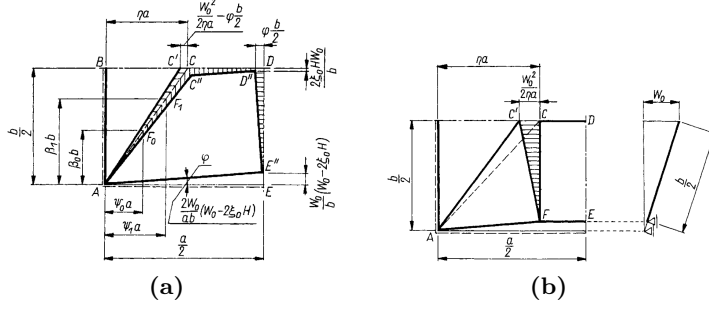


Figure 6.8: The collapse mechanism after Sawczuk and Winnicki [1965]

laterally restrained slabs, the static load-deflection relation is obtained with a method based on the flow theory of rigid-plastic bodies, from the initial compression arching action phase up to the tensile membrane stage. The author applied a kinematic approach with the initial collapse mechanism as assumed in the yield line theory (Figure 6.9) and preserved at large deflection, using an energy and an equilibrium method.

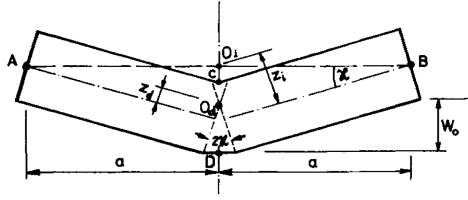


Figure 6.9: Yield hinges at large deflection in concrete slab (Janas [1968])

Hayes [1968] presented an equilibrium method for allowing for membrane action in predicting the plastic behaviour of orthotropically reinforced rectangular concrete slabs, in which the criticisms of Sawczuk's method are met and a closer tie with the solution of Kemp is achieved. Hayes considered two possible stress distributions in the section along the yield lines, depending on whether or not cracks penetrate to the upper surface of the slab, as illustrated in Figure 6.10.

The method proposed by Hayes uses a rigid, perfectly-plastic slab model, in which the collapse mechanism is defined by the yield line theory. The enhancement of the load-carrying capacity of the slab for larger deflections after the achievement of the ultimate load is due to the increase of the internal lever arm. Once the in-plane forces distribution along the yield lines is calculated, the load-bearing capacity of the slab is computed by considering moment equilibrium about the axis of rotation of the rigid slab elements.

Similarly to Hayes, Bailey [2001], evaluated the load-bearing capacity of orthotropic laterally unrestrained slabs with only one layer of reinforcement by means of an equilibrium method. Contrary to Hayes, Bailey included in

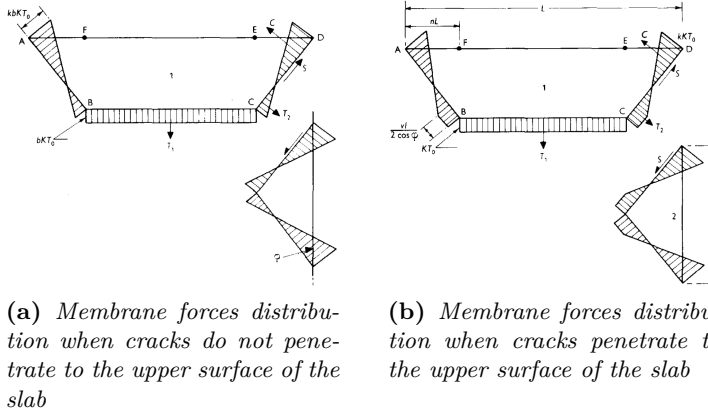


Figure 6.10: In-plane forces distribution for the equilibrium method after Hayes [1968]

the collapse mechanism the formation of an axial crack perpendicular to the longer side of the slab at its mid-span as shown in Figure 6.11. The force distribution along the yield lines and along the axial crack at mid-span is illustrated in Figure 6.12.

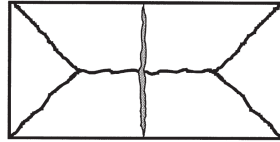


Figure 6.11: Failure mode after (Bailey [2001])

Omer [2006] proposed an analytical method for evaluating the load-deflection response of lightly reinforced concrete slabs restrained and unrestrained simply supported along the edges, considering both ambient and elevated temperatures. The author considered only one layer of reinforcement and included the effects of the bond strength and slip length between the concrete and the steel on the computations. The method proposed by the author considers a rigid, perfectly plastic kinematic model compatible with the collapse mechanism as shown in Figure 6.13, in which cracks formed as in the yield line theory. The principle of virtual work is used for the calculations in which the external work done by the external loads is equated to the energy dissipated by the members. The internal equilibrium of the slab is neglected in this approach. This method ignores the compressive membrane action and bending moments, and only considers the work done by the tensile membrane forces through a bond-slip relation, including the strain hardening of the reinforcement steel. The failure of the slab is defined as the point where the steel stress at the crack face reaches the ultimate

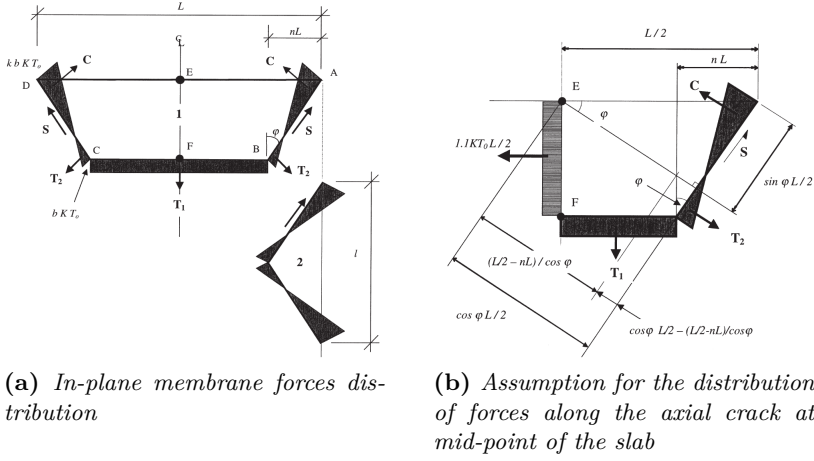


Figure 6.12: In-plane forces distribution for the equilibrium method after Hayes [1968]

strength of the material. As depicted in Figure 6.14, it is clear that the steel reinforcement slips out gradually from the concrete block towards the pulled end (Omer [2006]). The concentration of strain in the steel will be more at the concrete face, since the bond stress developed between the rebar and the surrounding concrete reduces the strain in the steel away from the pulled end. Therefore, it is assumed that the bar will fail at the concrete face once the fracture strain of the steel (and the ultimate strength of the material) is achieved.

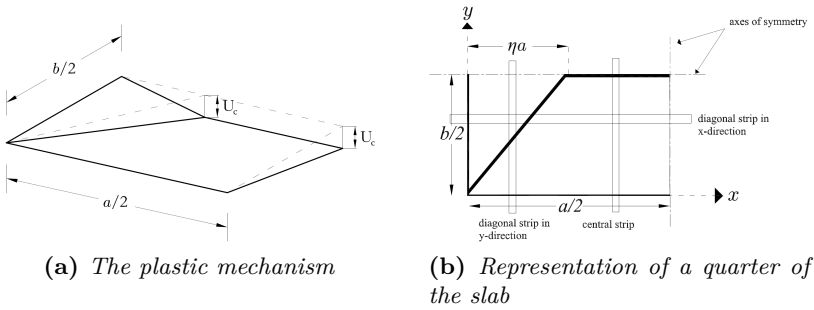


Figure 6.13: The plastic collapse mechanism after Omer [2006]

Once the kinematic model of the slab is assumed, the shortenings and the elongations of the members can be computed by simply geometrical considerations. The selection of constitutive relation of the material allows determining the forces in the members, and, consequently, the internal work. The forces in the rebars require the definition of a suitable bond stress be-

tween steel and concrete as illustrated in Figure 6.15 that can be evaluated for instance by means of experimental tests. Omer demonstrated that a value of $\sigma_b = 0.25 \text{ N/mm}^2$, where σ_b is the bond stress, can be assumed for the computations. The basic assumptions of the model are validated against non-linear analyses, obtaining a favourable comparison.

Even if the method proposed by Omer (Omer [2006]) is an efficient tool for the failure assessment of horizontally restrained lightly reinforced concrete slabs, some assumptions, such as only one layer of reinforcement, limit its use.

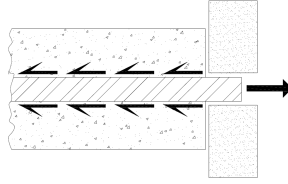


Figure 6.14: Representation of rebars that slip out of a concrete block (Omer [2006])

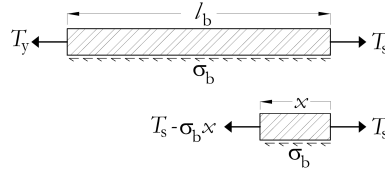


Figure 6.15: The influence of bond stresses developed along the rebars (Omer [2006])

Herraiz and Vogel [2016] proposed a novel design approach for the static analysis of laterally unrestrained rectangular concrete slabs with orthotropic reinforcement in one or two layers under a uniformly distributed load, considering membrane action. The method estimates the complete structural response, including load-bearing capacity, vertical deflections and other kinematic quantities (Herraiz Gómez [2016]), from the initial elastic phase to the last membrane phase. The behaviour of the slab prior to the development of membrane forces is estimated through classic, renowned methods, and the vertical deflection at which membrane action begins, by means of a perfectly-plastic kinematic model (Herraiz and Vogel [2016]). For larger deflections, the distribution of membrane forces is evaluated by means of an iterative procedure, satisfying both kinematics and equilibrium of the slab. The slab can be simply supported or rotationally restrained along the edges, but must be vertically supported. The position of the neutral axis along the slab edges d_c (e.g. the position of the axis of rotation) depends

on the rotational restraints and is expressed in term of a coefficient α as:

$$d_c = \alpha \cdot h \quad (6.1)$$

where h is the thickness of the slab. For simply supported slabs, the axis of rotation can be assumed at the mid-plane level of the slab ($d_c = 1/2h$) (Herraiz Gómez [2016]).

The kinematic model used by Herraiz, shown in Figure 6.16, is a rigid, perfectly-plastic slab model in which the cracks develop along the diagonals, the central region and the perpendicular line to the longer side of the slab in the mid-zone. The position of the neutral axis along the yield lines is assumed at the intersection of the rotated section with the vertical cross-section of the slab, and varies linearly along the yield line.

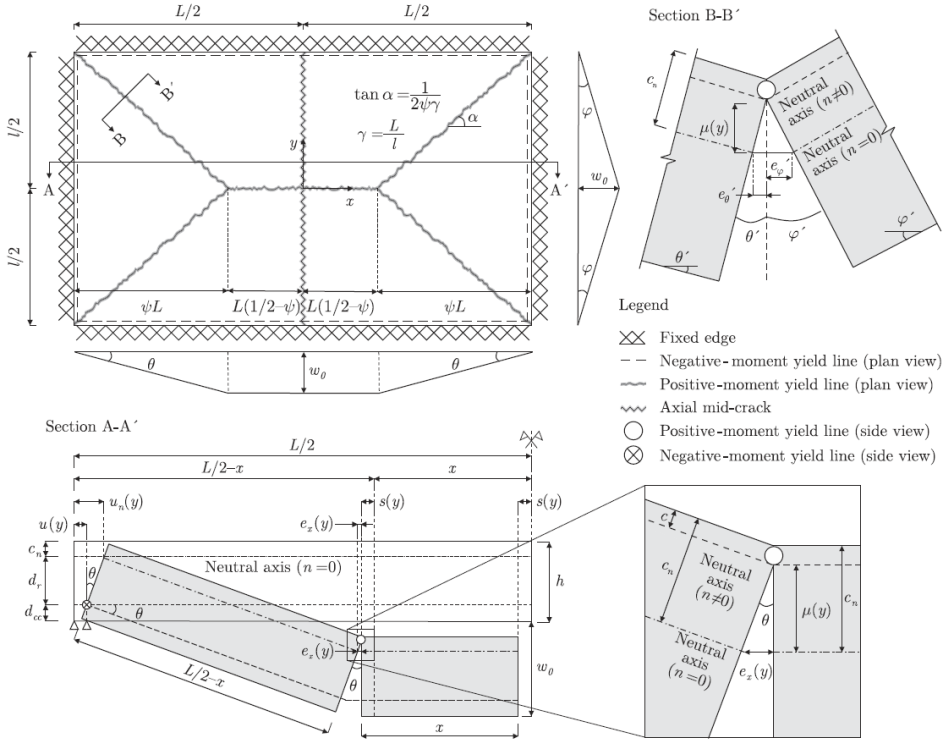


Figure 6.16: The rigid plastic slab mechanism after (Herraiz and Vogel [2016])

6.4 An analytical method to predict the response in membrane stage of laterally restrained reinforced concrete slabs

6.4.1 Introduction

In this section, an analytical method for the prediction of the static load-deflection response of reinforced concrete slab at large displacement is presented. This method takes advantage from the previous method developed by Omer [2006] at the Imperial College London, taking also into account some assumptions of Herraiz Gómez [2016]. The new approach is valid for reinforcement concrete square slabs with two layer of reinforcement simply supported and laterally restrained along the edges. This choice was made with the purpose of studying analytically the response of the *symmetric* framed structure tested at the University of Trento within the activities of the 'RobustImpact' research project and described in Chapter 4. The test was conducted on a full-scale steel-concrete composite structure in which the solid slab was connected with the steel beams with studs to reproduce the full shear composite action. The frame was one floor two-by-two-bay, extracted from a reference five-story full-frame building. During the test, the central column removal was simulated by means of an hydraulic actuator. In this method, the presence of the steel beams, and thus of the composite action, is ignored. The concrete slab is assumed as simply supported along the edges and subjected to a large displacement caused by the central column loss.

The membrane stage developed at large deflections is considered in the method, and the previous elastic and flexural stages are ignored. This choice is related to the fact that structures subjected to column removal exhibit large deflection of the slab system. As described in details in Chapter 4, several loading steps occur during a column loss event. First of all, the gravity loads act on the structure and define the conditions before the column loss (*step 1*); consequently, the collapse of the column occurs and important vertical displacements of the slab system develop (*step 2*); the final step (*step 3*) allows quantifying the residual strength of the structure and the development of the membrane stage in the slab. At the end of *step 2* the axial force in the removed column is null because the column is completely lost. For this purpose, this method is valid for the evaluation of the slab response during *step 3*.

Before the description of the method, the assumptions and the kinematic model of the slab are presented.

6.4.2 Assumptions

For reinforced concrete slabs restrained along the edges and subjected to a uniformly distributed load, the strain concentrations are located in correspondence of the diagonals and in the central region of the slab, as assumed in the yield line theory and demonstrated by several experimental analyses conducted by Foster et al. [2004] and depicted in Figure 6.17.

In these analyses the corner effects are ignored since they complicate the geometry of the yield mechanism; the central crack, as shown in Figure 6.17, does not form in square slabs, and the collapse mechanism associated with this configuration is the simple diagonal pattern illustrated in Figure 6.18. However, since the formation of single cracks along the diagonals leads to an increase of the strain in the crack face compared to the case with diffused cracks, the adoption of a single crack allows obtaining a conservative model of failure by fracture of the rebars (Omer [2006]).

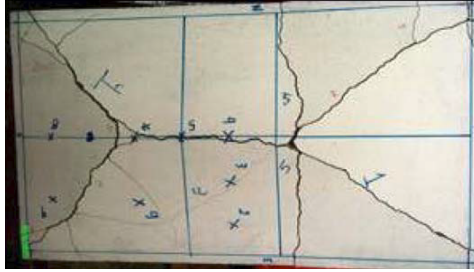


Figure 6.17: Bottom side of a slab subjected to large displacement (Foster et al. [2004])

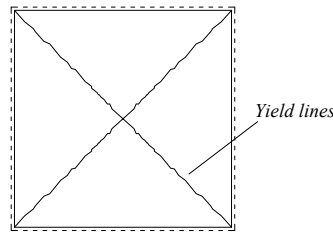


Figure 6.18: The assumed yield lines pattern in reinforced concrete square slabs

The following assumptions with reference to Figure 6.19 are the basis of the model:

- the slab is square and simply supported along the edges;
- the lateral edges are restrained against horizontal movements;
- the slab is reinforced with two layer of rebars, one in the bottom face and one in the top face;

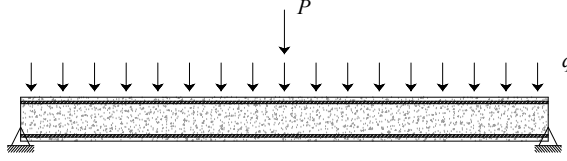


Figure 6.19: Schematic representation of the slab section

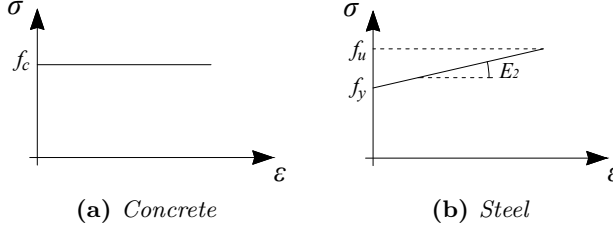


Figure 6.20: Material model adopted in the model

- the reinforcement in the two directions of the slab is assumed at the same level;
- the slab is subjected to a mid-span point load and to a uniformly distributed load. The point load is representative of the force acting in the central removed column for the evaluation of the residual strength of the system during the membrane stage;
- a rigid-plastic kinematic model is assumed to be compatible with the collapse mechanism as shown in Figure 6.18, in which the yield lines are developed along the diagonals of the slab;
- the triangular panels of the slab bounded by the yield lines rotate rigidly around the yield lines and the supports;
- rigid perfectly-plastic material is adopted for concrete as in Figure 6.20 (a). The tension strength of the concrete is neglected;
- rigid-plastic with hardening material is adopted for reinforcing bars steel as in Figure 6.20 (b).

The model is developed using a kinematic method of analysis (Calladine [2000]) by means of the principle of virtual work where the work done by the external loads W_e is equated to the internal work done by the structural members W_i for every deflection increment of the system.

$$W_e = W_i \quad (6.2)$$

In order to evaluate the static response of the slab subjected to column loss, some difficulties have been overcome during the developing of the

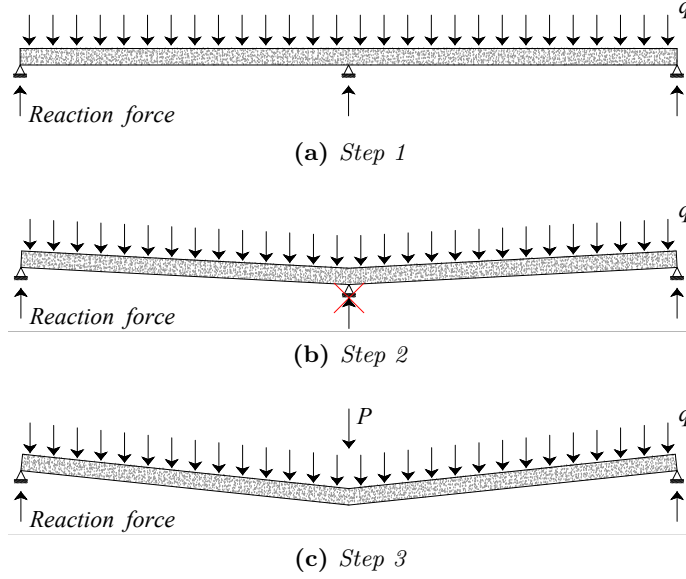


Figure 6.21: Representation of the loading steps during the column removal scenario

method. In a typical and general case, the structure, before the column collapse event, is completely loaded by gravity loads acting onto the floor system, as in *step 1* of the tests (see Chapter 4) depicted in Figure 6.21 (a). Furthermore, the column is removed (Figure 6.21 (b)) and the slab system exhibits large vertical displacements (*step 2*). Finally, a tensile force is applied in the removed column as shown in Figure 6.21 (c) for the evaluation of the residual strength (*step 3*). The uniformly distributed loads remain constant during all the loading steps.

The collapse mechanism is assumed to be formed when the central point of the slab achieves a deflection equal to the slab thickness. This choice was made with reference to the experimental results reported in §4.2, where the displacement measured at the end of the column removal was of about 160 mm, and to the studies of Herraiz Gómez [2016]. First of all, an initial deflection equal to the slab thickness is assigned to the slab and the uniformly distributed load, for which the external and internal work are equated, is computed. Consequently, the deflection is incremented and the point load at the centre of the slab is calculated.

6.4.3 Description of the kinematic model

The kinematic model of the concrete slab assumed in this study is a plastic model valid for the post-yielding response of the slab, allowing free rotation and non-deformable slab regions. This model ignores the pre-yielding

response of the slab, during which elastic strains develop. Studying the slab at large deflections, for which the method is developed, allows ignoring the elastic strains that are significantly smaller than the strains developed during the plastic membrane stages. The kinematic model is illustrated in Figure 6.22. The parameter α defines the position of the axis of rotation along the supports. For simply supported slabs, the slab rotates about the axis of the slab's mid-plane, and so $\alpha = 1/2h$ (Herraiz Gómez [2016]). The parameter n describes the position of the neutral axis of the cross section in correspondence of the yield line, and is assumed to be equal on both sides of the yield line.

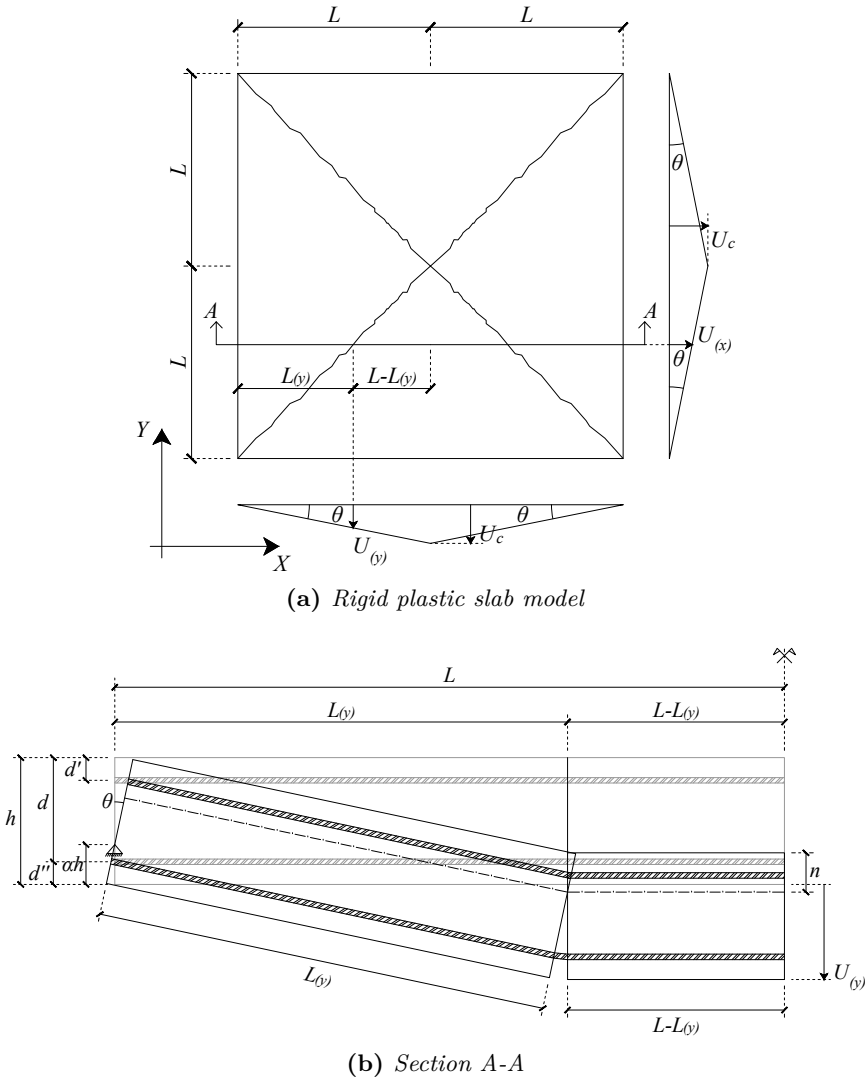


Figure 6.22: Representation of the assumed kinematic model

Giving a rotation of the section θ , the vertical displacements at the centre of the slab U_c and at the yield line sections $U_{(y)}$ can be computed as:

$$U_c = L \cdot \sin \theta + \alpha h \cdot \cos \theta - \alpha h \quad (6.3)$$

$$U_{(y)} = L_{(y)} \cdot \sin \theta + \alpha h \cdot \cos \theta - \alpha h \quad (6.4)$$

$$U_{(x)} = L_{(x)} \cdot \sin \theta + \alpha h \cdot \cos \theta - \alpha h \quad (6.5)$$

where $L_{(y)}$ and $L_{(x)}$ are the length of each strip from the support to the yield line section along the two slabs directions. For square slabs, the deflection $U_{(y)}$ is equal to the deflection $U_{(x)}$.

The position of the neutral axis n is geometrically determined as a function of the rotation θ as:

$$n = \frac{L_{(y)}}{\tan \theta} - \frac{L_{(y)}}{\sin \theta} + h - \alpha h \quad (6.6)$$

6.4.4 Evaluation of the load-deflection response

The load-deflection response of the slab is determined with a step-by-step procedure using a kinematic method of analysis where the work done by the external loads is equated to the work done by the structural elements. For this purpose, the individual contributions (see Equation 6.7) of the reinforcing bars, of the concrete, and of the external loads are analysed separately in the following. Due to symmetry, only a quarter of the slab can be considered, as shown in Figure 6.23. The strips $d_{s,x}$ and $d_{s,y}$ through the length and width of the slab can be idealized as reinforced concrete beams in the two main directions X and Y of the slab.

$$W_e = W_{i,s} + W'_{i,s} + W_c \quad (6.7)$$

Depending on the position of the neutral axis directly related to the rotation of the section, three cases can be assumed:

- the neutral axis is between the two layers of reinforcement. The rebars in the bottom layer are in tension, while the rebars in the top layer are in compression, as illustrated in Figure 6.24 (a). The concrete below the neutral axis is assumed to be cracked due to neglecting the tensile strength of the material;
- the neutral axis is between the top layer of reinforcement and the top side of the slab as in Figure 6.24 (b). In this case the bottom rebars remain in tension, while the upper rebars pass from compressed to stretched;

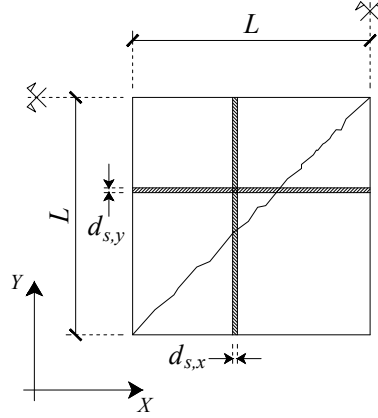


Figure 6.23: Representation of a quarter of the slab

- the neutral axis is above the top side of the slab (Figure 6.24 (c)). A full depth crack is formed and the concrete is completely cracked. Its contribution is therefore neglected in this phase.

Reinforcing bars - bottom layer

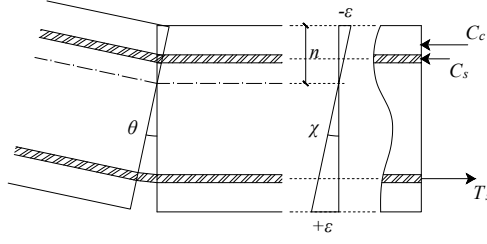
The response of the reinforcing bars of the bottom layer is obtained by adding the contribution of all the strips in the X and Y directions of the slab. Assuming the slab as squared and isotropically reinforced, the response of the rebars in the two directions are equal between them. For this purpose, the strips in the X direction are considered in the following as depicted in Figure 6.25 (a).

Referring to Figure 6.25 (b) and Figure 6.24, the elongation of the reinforcements $\Delta_{s,x}$ can be calculated as:

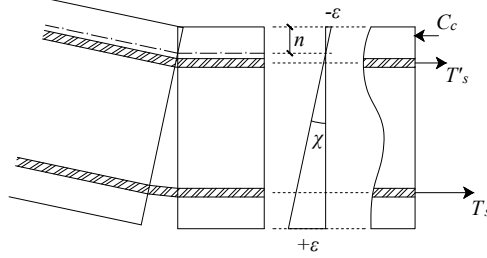
$$\Delta_{s,x} = 2(d - n) \sin \theta \quad (6.8)$$

The position of the neutral axis n is described by Equation 6.6. The tensile force acting in the rebars is calculated by means of the constitutive relation of the material and the bond strength between steel and concrete, following the development of Omer [2006]. In particular, Omer assumed that the stretching of the bars is located in a region where the steel stress exceeds yield, due to the assumption of the rigid-hardening material. The force $T_{s,x}$ at the crack face is greater than the yielding force $T_{y,x}$ at a distance l_b from the crack face, according to the bond strength σ_b , as depicted in Figure 6.15 (Omer [2006]). Therefore, l_b can be described as:

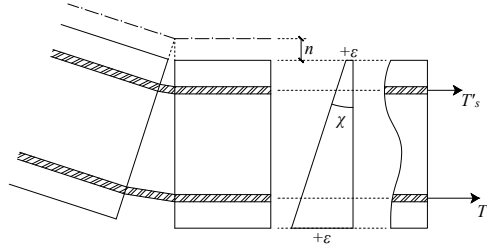
$$l_b = \frac{T_{s,x} - T_y}{\sigma_b} \quad (6.9)$$



(a) Neutral axis between the two layer of reinforcement

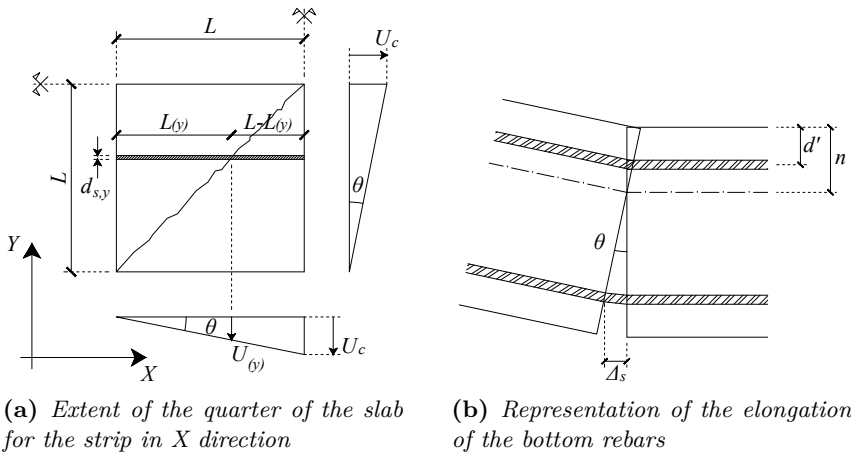


(b) Neutral axis between the top layer of reinforcement and the top side of the slab



(c) Neutral axis above the top side of the slab

Figure 6.24: Representation of the three cases of the position of the neutral axis



(a) Extent of the quarter of the slab for the strip in X direction

(b) Representation of the elongation of the bottom rebars

Figure 6.25: Schematic representation of the strips in X direction

At a distance x from the crack face, the force $T(x)$ and the strain ϵ_s are:

$$T(x) = T_{s,x} - \sigma_b x \quad (6.10)$$

$$\epsilon_s = \frac{T(x) - T_{y,x}}{A_{s,x} E_2} \quad (6.11)$$

where $A_{s,x}$ is the cross section of the reinforcement in the X direction expressed per unit width of the slab and E_2 is the hardening modulus of the steel as depicted in Figure 6.20. The term $T(x) - T_y$ denotes the increment of the force over the yielding force. The extension of the bars Δ_s is obtained by integrating within strains in the bond length l_b as:

$$\Delta_{s,x} = \int_0^{l_b} \epsilon_s dl = \frac{T_{s,x} - T_{y,x}}{A_{s,x} E_2} l_b - \frac{\sigma_b l_b^2}{2 A_{s,x} E_2} \quad (6.12)$$

Substituting Equation 6.9 into Equation 6.12, Δ_s becomes:

$$\Delta_{s,x} = \frac{(T_{s,x} - T_{y,x})^2}{2 A_{s,x} E_2 \sigma_b} \quad (6.13)$$

Isolating force $T_{s,x}$ in the Equation 6.13, the force in the reinforcement at the crack face is obtained in terms of $\Delta_{s,x}$, as (Omer [2006]):

$$T_{s,x} = T_{y,y} + \sqrt{2 A_{s,x} E_2 \sigma_b \Delta_{s,x}} \quad (6.14)$$

It is important to notice that the force, the cross section, and the bond strength are expressed per unit width of the slab.

The work done by the bottom reinforcement in the X direction is obtained by the following integral:

$$W_{i,s,x} = \int_0^L T_{s,x} \Delta_{s,x} d_{s,y} \quad (6.15)$$

With reference to Figure 6.26, the work done by the bottom reinforcement in the Y direction can be computed as:

$$W_{i,s,y} = \int_0^L T_{s,y} \Delta_{s,y} d_{s,x} \quad (6.16)$$

Reinforcing bars - top layer

The response of the rebars of the upper layer of reinforcement depends on the position of the neutral axis as described previously and as illustrated in Figure 6.24. In particular, the upper rebars are compressed in the beginning of the analysis, and become stretched for an increased deflection of the slab. For this purpose, the constitutive model of the material assumed for the

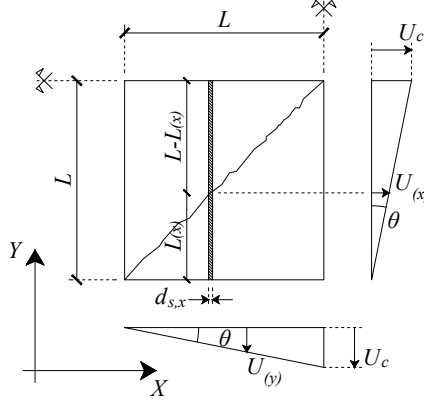


Figure 6.26: Schematic representation of the strips in Y direction

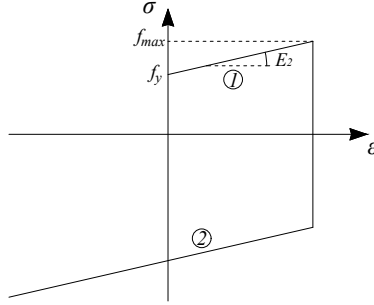


Figure 6.27: Constitutive relation assumed for the steel of the upper rebars

upper rebars is shown in Figure 6.27, where '1' denotes the initial compressed phase and '2' the distress and tensile phases.

The response of the upper rebars is obtained with reference to the procedure described for the bottom rebars. The shortening of the bars (as shown in Figure 6.28) is geometrically calculated as:

$$\Delta'_{s,x} = (n - d') \tan \theta \quad (6.17)$$

Two cases can be identified:

- case a) if $n > d'$ (Figure 6.28 (a)), the upper rebars are compressed;
- case b) if $n < d'$ (Figure 6.28 (b)), the upper rebars are distressed and become tensioned for further deflections.

The force acting in the rebars in the X direction during the compressed phase a) is obtained by the following equation:

$$C_{s,x} = C_{y,x} + \sqrt{2A'_{s,x} E_2 \sigma_b \Delta'_{s,x}} \quad (6.18)$$

where $C_{y,x}$ is the yielding force of the rebars in the X direction expressed per unit width of the slab. During the tensile phase b), the force becomes:

$$C_{s,x} = -C_{s,x,max} - \sqrt{2A'_{s,x}E_2\sigma_b(\Delta'_{s,x,max} - \Delta'_{s,x})} \quad (6.19)$$

The maximum compressive force $C_{s,x,max}$ is relative to the achievement of the maximum shortening $\Delta'_{s,x,max}$ that is numerically computed. The shortening and the forces acting in the rebars in the Y direction are expressed as:

$$\Delta'_{s,y} = (n - d') \tan \theta \quad (6.20)$$

$$C_{s,y} = C_{y,y} + \sqrt{2A'_{s,y}E_2\sigma_b\Delta'_{s,y}} \quad (6.21)$$

$$C_{s,y} = -C_{s,y,max} - \sqrt{2A'_{s,y}E_2\sigma_b(\Delta'_{s,y,max} - \Delta'_{s,y})} \quad (6.22)$$

The work done by the reinforcement of the upper layer in the X and in the Y directions, respectively, is given by the following equations:

$$W'_{i,s,x} = \int_0^L C_{s,x} \Delta'_{s,x} d_{s,y} \quad (6.23)$$

$$W'_{i,s,y} = \int_0^L C_{s,y} \Delta'_{s,y} d_{s,x} \quad (6.24)$$

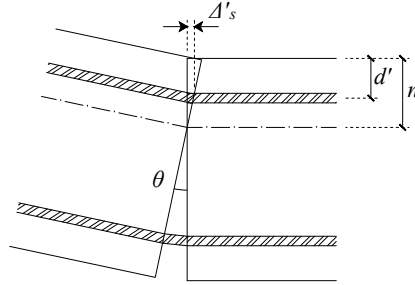


Figure 6.28: Representation of the shortening of the upper rebars

Concrete

The shortening of the concrete Δ_c is obtained with reference to Figure 6.29, where z denotes the depth of the concrete from the top face of the slab. In particular, z can be expressed by an array from 0 to n for each step of the analysis and for each strip of the slab.

Known the concrete depth z , the shortening of the concrete is computed as:

$$\Delta_c = z \cdot \tan \theta \quad (6.25)$$

Since rigid perfectly-plastic behaviour of the concrete is assumed, as shown in Figure 6.20, the stress is equal the the compressive strength of the concrete f_c .

The work done by the concrete can be obtained by integrating over the region z as:

$$W_{i,c} = f_c \cdot d_{s,y} \int_0^L \int_0^z \Delta_c dz ds_y \quad (6.26)$$

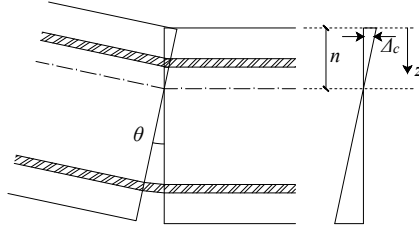


Figure 6.29: Representation of the shortening of the upper rebars

Therefore, the overall internal work done by the members across the yield lines is obtained from:

$$W_i = 4 (W_{i,s,x} + W_{i,s,y} + W'_{i,s,x} + W'_{i,s,y} + W_{i,c}) \quad (6.27)$$

External loads

The work done by the external point load acting at the centre of the slab is equal to:

$$W_{e,P} = P \cdot U_c \quad (6.28)$$

Assuming that the slab panels rotate rigidly about the yield lines, the work done by the uniformly distributed load is equal to q multiplied by the volume created by the slab as it deforms (Omer [2006]). In particular, referring to the quarter of the slab, the work is equal to:

$$W_{e,q} = \frac{q \cdot L^2 \cdot U_c}{3} \quad (6.29)$$

Combining Equations 6.27, 6.28 and 6.29, a relationship between the point load and the deflection of the slab can be obtained by means of the principle of virtual work (Equation 6.2) as:

$$P = \frac{W_i}{U_c} - \frac{4 \cdot q \cdot L^2}{3} \quad (6.30)$$

6.5 Application of the method to a case-study slab and verification with a numerical tool

Implementation of the method

The analytical method presented in section 6.4 is applied here to a case study slab and compared against the finite element analyses software Abaqus (Simulia Dassault Systems). The slab of the *symmetric* sub-frame tested at the University of Trento and described in Chapter 4 is assumed for the analyses. The analytical method is implemented in the numerical software MATLAB (The MathWorks [2017a], The MathWorks [2017b]). The geometric properties of the slab, as shown in Figure 6.30, are reported in Table 6.1, while the properties of the materials in Table 6.2.

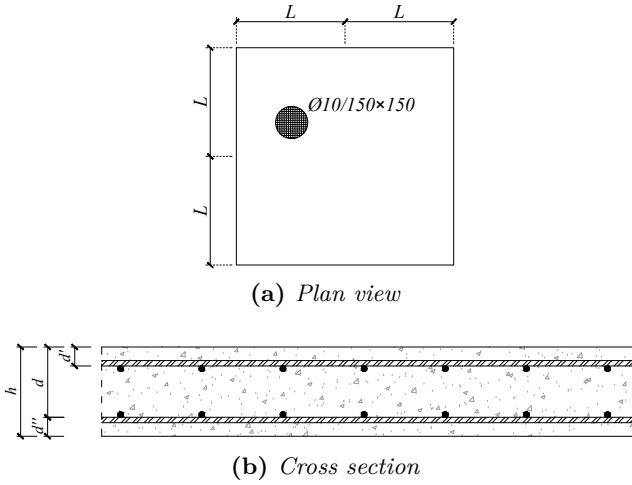


Figure 6.30: Representation of the geometric properties of the slab

Table 6.1: Geometric properties of the slab

$L = 5700$ mm	$A_{s,x} = 0.52$ mm ² /mm ($\phi 10/150$)
$h = 165$ mm	$A_{s,y} = 0.52$ mm ² /mm ($\phi 10/150$)
$d = 130$ mm	$A'_{s,x} = 0.52$ mm ² /mm ($\phi 10/150$)
$d' = 35$ mm	$A'_{s,y} = 0.52$ mm ² /mm ($\phi 10/150$)
$d'' = 35$ mm	

The main aim of the method is to study the response of the slab after the column loss. As described previously, the yield mechanism is assumed as formed when the deflection at the centre of the slab is equal to the slab thickness. Until this point, the slab is subjected to the uniformly distributed load only. Afterwards, the point load is applied at the centre of the slab increasing the deflection. For this purpose, a deflection of $U_c = 165$ mm is

Table 6.2: Mechanical properties assumed for the materials and for the bond-strength

$E_2 = 850 \text{ MPa}$
$f_y = 450 \text{ MPa}$
$f_u = 550 \text{ MPa}$
$f_c = 50 \text{ MPa}$
$\sigma_b = 0.25 \text{ N/mm}^2$

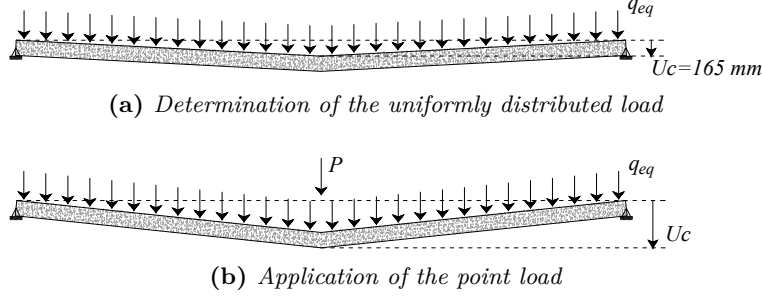


Figure 6.31: Schematic representation of the phases implemented in the method

applied to the slab (Figure 6.31 (a)) and an uniformly distributed load q_{eq} such that the external work is equal to the internal work is computed. The load q_{eq} is obtained by means of the principle of virtual work described by the Equation 6.2, where the external work is generated by the uniformly distributed load only, without the contribution of point load P . In particular, the load q_{eq} is:

$$q_{eq} = \frac{3 \cdot W_i}{4 \cdot L^2 \cdot U_c} = 14.7 \text{ kN/m}^2 \quad (6.31)$$

Consequently, point load P is applied at the centre of the slab for further increased deflections (Figure 6.31 (b)). The load-deflection relationship is obtained by means of Equation 6.30 with a step-by-step procedure, where the uniformly distributed load q is equal to q_{eq} as:

$$P = \frac{W_i}{U_c} - \frac{4 \cdot q_{eq} \cdot L^2}{3} \quad (6.32)$$

The finite element model of the slab

The slab (Figure 6.32) is modelled with 2D shell elements as in the finite element model of the sub-frames described in Chapter 4. The horizontal and vertical displacements along the slab edges are restrained, while the rotations are left free. The slab is meshed with 4-node stress-displacement shell elements ('S4R') with sides of approximately $250 \times 250 \text{ mm}$. The

reinforcements are implemented in the slab as embedded and according to the geometric properties reported in Table 6.1. The '*Concrete Damaged Plasticity*' model is adopted for the concrete with the properties shown in Table 4.11, while the steel is modelled with an elastic-plastic with hardening law as described in Table 6.2.

In order to simulate the central column collapse, a vertical elastic beam element is introduced and connected with the centre of the slab simulating the presence of the column. For this purpose, the analysis is conducted with the following sequence of phases:

- *step 1*: the uniformly distributed load $q_{eq} = 14.7 \text{ kN/m}^2$, obtained from the analytical method solution, is applied onto the slab and the reaction force in the central column is carried out;
- *step 2*: a force equal and opposite to the reaction force is applied to the central column in order to simulate the column loss;
- *step 3*: a point load at the centre of the slab is applied and increased up to the end of the analyses.

The load-deflection response of the slab during *step 3* is illustrated in Figure 6.33. When the column is completely removed, the deflection at the centre of the slab obtained from the FEM model is greater than the slab thickness as assumed in the analytical model. During the application of the point load, the comparison of the results seems to be satisfactory. Though the strains in the analytical model are located along the diagonals and the slab panels rotate rigidly about the axis of rotation, the concrete model implemented in the FEM analyses allows developing diffused strains on the slab surface. Anyway, the vertical displacements of the slab illustrated in Figure 6.34 show a sort of pyramidal deformed shape, allowing the kinematic model of the method to be verified. The slab modelled with shell elements and the reinforcement as embedded in the slab does not allow checking all the kinematic quantities of the analytical method, such as the elongation and the shortening of the rebars and of the concrete in the section along the diagonals.

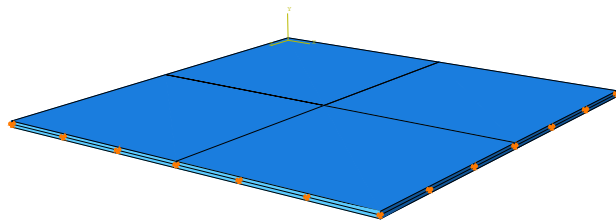


Figure 6.32: *The finite element model of the slab*

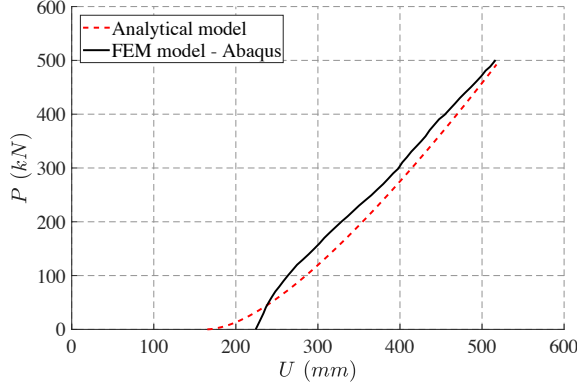
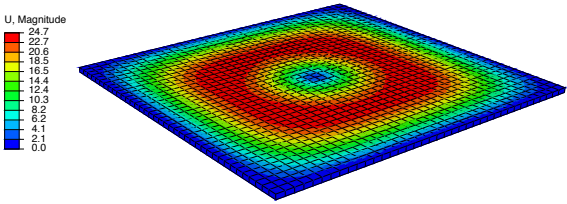


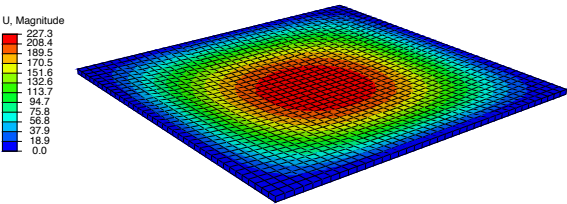
Figure 6.33: Load-deflection response of the slab

6.6 Concluding remarks

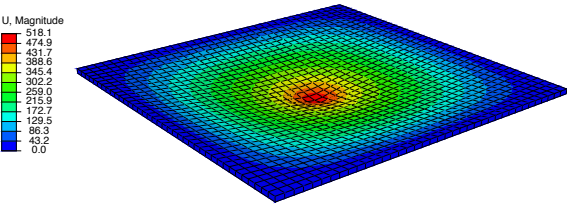
An analytical method for the evaluation of the static load-deflection response of a simply supported reinforced concrete slab subjected to large deflections, resulting from a column loss, has been presented in this chapter. The method based on the studies of Omer [2006] and Herraiz Gómez [2016], is developed using a kinematic method of analyses by means of the principle of virtual work where the work done by the external loads is equated to the internal work done by the structural members for each deflection increment of the slab. The model is validated against a finite element analysis conducted with Abaqus for the isolated square slab tested at the University of Trento described in the previous chapters. The results showed a satisfactory comparison of the obtained curves, verifying the basic assumptions of the analytical method.



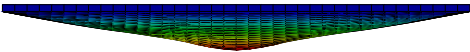
(a) Step 1



(b) Step 2



(c) Step 3



(d) Front view - step 3

Figure 6.34: Vertical displacement of the slab (units in mm)

Chapter 7

General conclusions and further research

The main aim of this research is to investigate the performance of steel-concrete composite structures subjected to catastrophic events. Several catastrophic events that occurred in the recent decades, demonstrate the need to design and build robust structures in order to limit the extent of the damage and protect the life of the occupancies. Among the various approaches recommended for designing robust structures, the alternate path has the advantage of conjugating simplicity and strength. A typical loading scenario for checking an alternate load transfer path is the one associated with the loss of a column.

In this chapter, the summary and conclusions drawn from each chapter of this work are presented and recommendations for further research are made.

7.1 Summary

The first three chapters of the thesis describe the motivations and objectives of the research and report the state of the art on structural robustness. After the collapse event of Ronan Point in 1968 in UK, the British design Standards started to include the topic of 'Robust design' in their recommendations. From that date, several authors studied the progressive collapse phenomenon and the measures to enhance the structural robustness of buildings. Experimental analyses on 2D or 3D frame structures and on single structural members, e.g. slabs and beam-to-column connections, have been conducted in order to characterize their behaviour in case of catastrophic events. Even if tests on three dimensional full scale specimens are the most complete and recommended, most of the experimental studies focused on the characterization of two-dimensional frames or even individual components. Furthermore, recently several authors numerically and analytically studied the column loss effects in building structures and the floor system

subjected to large displacement.

The column collapse causes important deformations of the structural members and dynamic effects. A robust response of the structure is achieved if the forces and loads can be efficiently redistributed from the damaged to the undamaged part of the structure. To this end, redundancy, ductility, and strength of the frame are fundamental requirements in order to mitigate the effect of catastrophic events. The contribution of the concrete slab is usually neglected, but, it was demonstrated that the development of membrane action in concrete slabs enhances the load carrying capacity and mitigate the effects of column loss.

At present, the design codes provide general strategies in order to design a structure for accidental events, underlining the fact that a significant revision of the codes is needed. In particular, specific guidelines and rules should be provided in the Codes to enable a reliable design approach. While research on concrete systems advanced already to a state adequate to develop such rules, other fields, such as for example steel-concrete composite frames, have not yet been adequately investigated. The work presented herein focuses on this area.

Experimental tests conducted at the University of Trento in the framework of the European research project 'RobustImpact' were used as the starting point of this research. Chapter 4 presented the tests and their principal results. In particular, tests on two full-scale three-dimensional composite steel-concrete sub-frames, extracted from the reference buildings, were conducted simulating the central column removal. The tests aimed to investigate the role of the floor system and of the beam-to-column connections in allowing the development of alternate load paths, preventing the collapse of the entire structure. The specimens achieved 'collapse' due to the rupture of two bolts in the bottom row of a joint connecting a beam with the removed column. At the end of the tests, a visual inspection of the slab revealed concentric compressive rings on the top side, typical of slabs where membrane forces are activated. A first general conclusions from the outcomes of the tests confirmed that the Eurocodes specifications, used to design the structures, seem a good base for the design of a robust composite frame.

The results of the tests allowed validating finite element models developed in the Abaqus/CAE environment. The numerical models were developed for both the sub-frames and full-frames object of the studies of the 'RobustImpact' research project. The comparison between the experimental and numerical results extended the informations gained from the tests, and enabled to get more data related to local strains and stresses, internal forces and other quantities that could not be measured during the tests. Two models of the steel beam-to-column connections were studied in order to check the results accuracy. In particular, the joints were modelled as pinned and as semi-rigid using the component method in the Eurocodes. The model with semi-rigid connections is more accurate than the model with pin connec-

tions, even if ideal hinges at the intersection of the beams with the columns is an effective mode to model the frames and predicting the overall response and force distribution.

The comparison between the results of the full-frame and sub-frame models showed that the mechanisms developed in the frames after the column loss and the load redistribution are similar, even if studying sub-structures is more conservative than studying the overall building.

The important contribution of the concrete slab on the behaviour of the overall system is confirmed and demonstrated by the comparison of the results of finite element models with and without the slab. With the same loading configuration, the vertical displacements of the frame without the slab are significantly greater than the displacements resulting from the model in which the slab is implemented.

Chapter 5 presented a study of the influence of the location of removed column on the structural response. Finite element analyses on both full-frame and sub-frame structures were developed simulating the loss of central, lateral, and corner columns. The results of corner column loss case showed that the ratio P/P_{max} between the force P acting in the removed column and the maximum reaction force P_{max} at the end of the analyses is significantly greater than the ratio in the case of internal or lateral column loss. Besides, the structural capacity of the sub-frames is smaller than the capacity of the full-frames due to the absence of the upper stories and of frame continuity. At the end of column removal, the columns close to the affected part of the structure are directly influenced by the event, supporting a load higher than the load acting before the column loss; the frames that are sufficiently far from the affected part are not influenced by the load redistribution.

Chapter 6 focused on the response of reinforced concrete slabs subjected to large displacements. The main aim of this study was to provide a tool for determining the response of concrete slabs in the membrane stage after the formation of the yield lines.

In the first part of the chapter, a short description of the main analytical studies carried out in the recent years to evaluate the behaviour of slabs in the membrane stage was presented, with a particular attention to the methods proposed by Omer [2006] and Herraiz Gómez [2016]. Omer developed a method for assessing the failure of composite steel-concrete slabs under ambient and elevated temperature by means of an energy approach, and Gomez analysed the static response of flat slab structures subjected to a sudden column failure scenario by verifying equilibrium and displacement compliance for each deflection increment of the slab.

The method presented in this chapter allowed evaluating the static load-deflection behaviour of simply supported reinforced concrete slabs subjected to large displacements, resulting from a column loss scenario. The method takes the start from the previous studies by Omer and Gomez, and is developed using a kinematic method of analysis by means of the principle of

virtual work, where the work done by the external loads is equated to the internal work done by the structure for each deflection increment of the slab. The method accounted for the concrete and for the steel of the bottom and upper reinforcing bars contribution, contrarily to the Omer method that accounts only for the presence of one layer of reinforcement. The collapse mechanism, as defined by the yield lines theory, was firstly defined, and the kinematic model assumed. The deformation of the members were computed by means of geometrical considerations and the forces calculated once the constitutive model of the material were defined.

Finally, the model was validated against a finite element analysis conducted with Abaqus/CAE environment for the isolated square slab with the overall geometry of the specimens tested at the University of Trento. The results showed a satisfactory comparison of the curves, verifying the basic assumptions of the analytical method and confirming the importance of membrane forces in enhancing the load-carrying capacity of slab systems.

7.2 Conclusions

The present work has presented experimental and numerical analysis on steel-concrete composite structures subjected to column removal. The behaviour of concrete slabs subjected to large displacement has also been investigated and an analytical tool for assessing the static load-deflection response of simply supported reinforced concrete slabs in which membrane actions are activated has been developed.

The literature review, reported in the first part of the dissertation, showed a lack of experimental tests on 3D full-frames subjected to extreme loading conditions, such as column loss. Experimental tests allow investigating the structural response of the system, on the collapse mechanism, on the load redistribution, on the activation of alternative load paths and on the residual strength of the frame. Furthermore, experimental data allow developing and validating finite element models, that can be used to carry out parametric analysis, enabling study of different structural systems, different loading conditions, and different design scenarios. The experimental tests presented in Chapter 4 showed the important role of the concrete slab for the force redistribution in the structure, and of the beam-to-column connections in order to avoid the progressive collapse after column removal.

In this context, the experimental results allow pointing out the following important considerations and design hints:

- the critical role of joints' ductility. The rotation of about 70 mrad should be achieved by the beam-to-column connections before rupture of the bolts. The composite joints designed as ductile under monotonic loading, enable development of the catenary action in the

beams. However, enhanced design of joints, taking into account local phenomena, such as local buckling of the bottom beam flange and web, and shear deformation of the column web panel is needed;

- an adequate anchorage of the slab to the lateral beams have to be adopted in design by means of shear connectors in order to improve slab efficiency by involving of these beams to the mechanism of resistance of the slab;
- cracks in the slab in behind the external columns showed the significant importance of the mechanism of force transmission from the slab to these columns. To allow an adequate mechanism of force transmission, detail rules for the design of the reinforcement bars, such as 'U' shaped rebars, have to be adopted. The model of Pardatscher [1996] seems to be effective to design these particular details.

The validated finite element models against experimental data allowed to develop further parametric numerical analysis for better understand the structural mechanisms. In particular, an important contribution has been given to the study of the influence of the location of the removed column on structural behaviour. Two scenarios are identified as important and opposite between them: the collapse of a central column and the collapse of a corner column. The first one allows activate a full membrane action in the concrete slab and alternative load paths for an efficient load redistribution. In this case, a robust structural response can be achieved if design hints, as described previously, are taken into consideration in the design of the system. The second one clearly pointed out the need of design key elements or overstrength of the structural members in order to sustain the large resistance demand and to avoid progressive collapse.

A further relevant contribution of this work is the study of membrane action in concrete slabs. The present work has presented a new simplified analytical method to predict the load-deflection response of simply-supported reinforced concrete slabs subjected to large displacements, that takes the start on the formulations of Gomez and Omer. The method takes into account both concrete and steel of the upper and bottom rebars, differently to Omer who studied slabs with one layer of reinforcement.

The numerical and analytical results obtained significantly contributed to the knowledge of the response of steel-concrete composite frames to accidental actions. Such a knowledge will support the development of specific guidelines for the design of robust composite frames. The potential further enhancement and use of the tools developed in this study are presented in the following section.

7.3 Further research

The finite element analysis of composite steel-concrete frame buildings subjected to column loss and the proposed analytical tool for assessing the load-deflection response of simply supported reinforced concrete slabs subjected to large displacements offers several research possibilities. The finite element models can be enhanced accounting for the beam-to-slab connection, and the failure criteria of the bolts in the joints model. Furthermore, the dynamic effects caused by the sudden column removal can be accounted for by developing dynamic/explicit analysis.

A possible further development of the analytical method could be the study of the influence of lateral restraints on the slab response; the slab is considered as restrained and simply supported along the edges. Furthermore, the analytical method can be enhanced including the presence of the steel beams in the floor system. In this context, an efficient tool accounting for the beam-to-column connections response is needed when the beams are included in the system and the composite action between steel and concrete is activated.

A further work could be the application of the method to a large set of existing slabs, with different geometrical properties, layouts and materials. In this sense, the accuracy of the method could be further validated.

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Appendex

Appendix A

Load redistribution in the structures

This Appendix provides the distribution of axial forces in the columns, both in the full-frame and sub-frame structures. In each figure, the values are reported at the end of the three steps: loading phase (*step 1*), column removal phase (*step 2*), deflection of about 300 mm (*step 3*). Furthermore, the variation of the force during *steps 2* and *3* is indicated with respect to the force acting at the end of *step 1* and are reported in Table A.1 and Table A.2 for the full-frame and sub-frame structures, respectively. The tensile and compressive stress distributions in the bottom and top side of the slab are also reported for each column removal scenario in the full-frame structures.

Table A.1: Axial force in the columns at the end of step 1 (units in kN) - full-frame

FRAME A							
	A1	A2	A3	A4	A5	A6	A7
Symmetric	471.5	1036.8	976.4	1011.9	956.1	1035.3	420.2
Asymmetric	501.4	1329.5	1256.7	1245.0	1214.2	1317.0	586.9
FRAME B							
	B1	B2	B3	B4	B5	B6	B7
Symmetric	1177.8	2476.8	2353.2	2178.4	2357.2	2461.6	1171.6
Asymmetric	1213.5	2545.7	2429.0	2369.2	2425.5	2511.5	1227.0
FRAME C							
	C1	C2	C3	C4	C5	C6	C7
Symmetric	491.8	1056.1	991.9	959.5	988.9	1051.2	487.9
Asymmetric	314.0	707.8	651.7	630.6	666.3	707.1	320.0

Table A.2: Axial force in the columns at the end of step 1 (units in kN) - sub-frame

	A1	A2	A3	B1	B2	B3	C1	C2	C3
Symmetric	76.4	227.4	91.5	233.7	711.1	370.7	76.1	226.3	92.5
Asymmetric	101.7	294.1	121.2	240.7	743.0	379.7	38.3	118.4	46.0

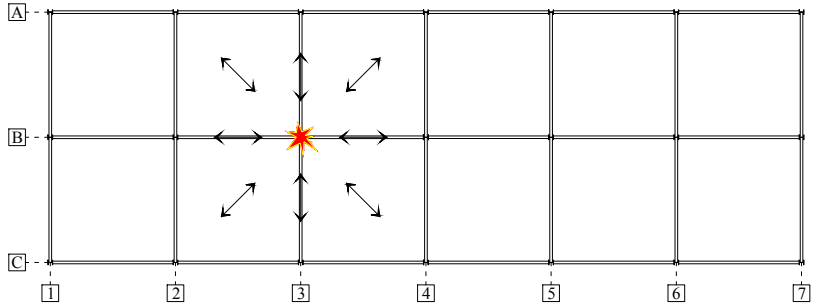


Figure A.1: Direction of the loads redistribution - column B3 removal

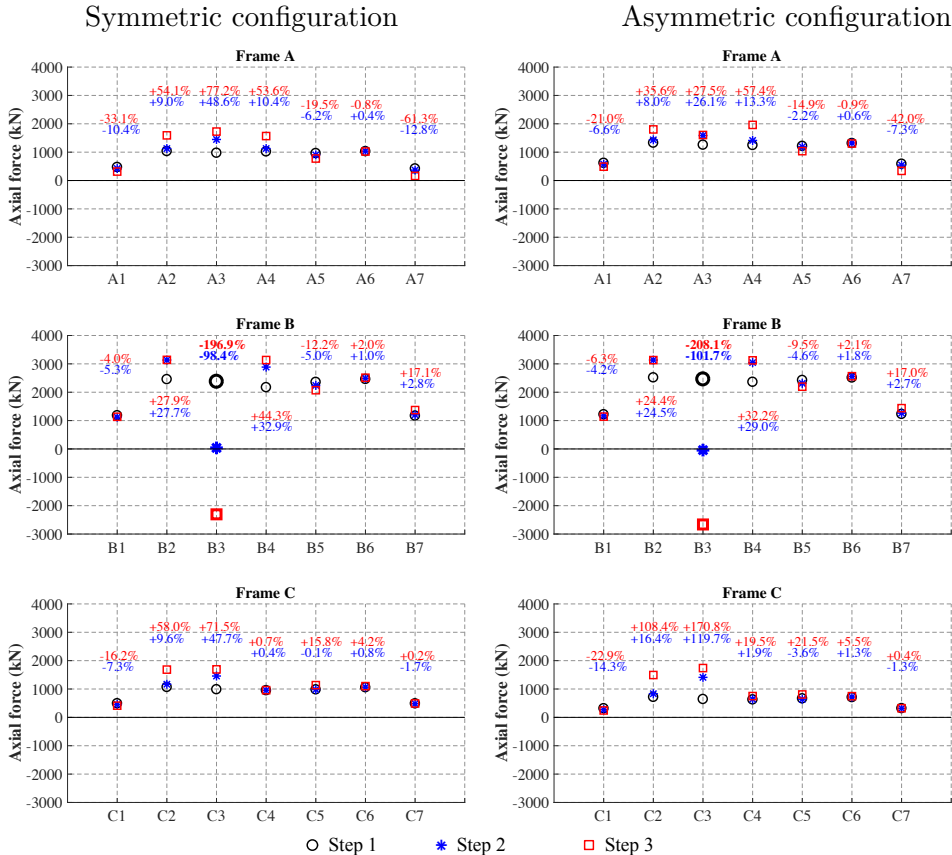
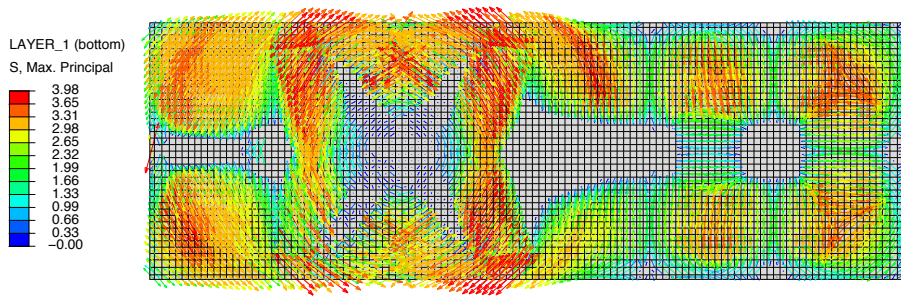
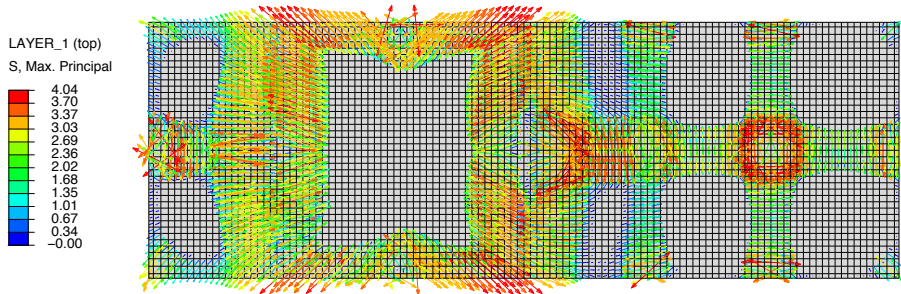


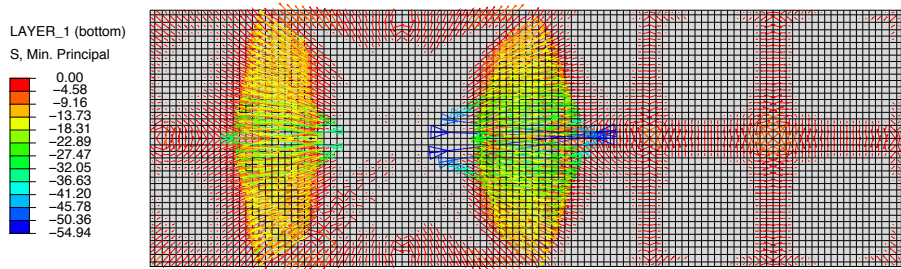
Figure A.2: Axial force in the columns - full-frame - floor 1 - column B3 removal



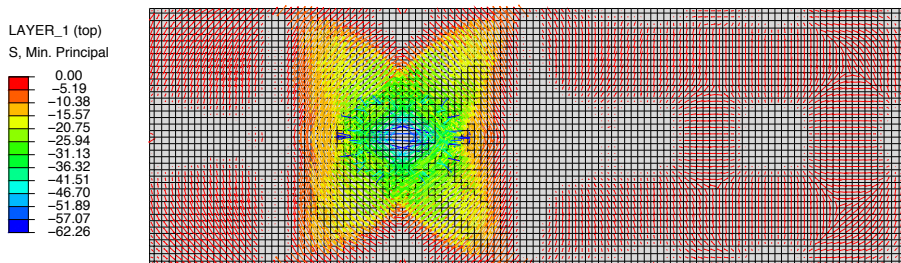
(a) Tensile stresses - bottom side



(b) Tensile stresses - top side

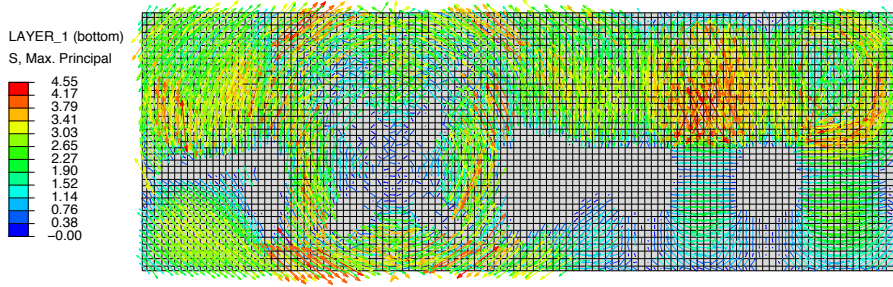


(c) Compressive stresses - bottom side

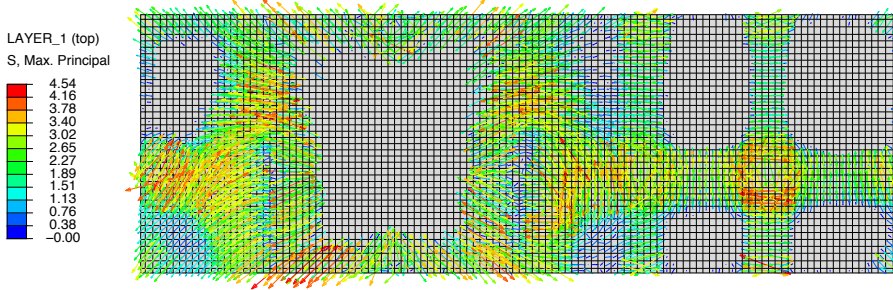


(d) Compressive stresses - top side

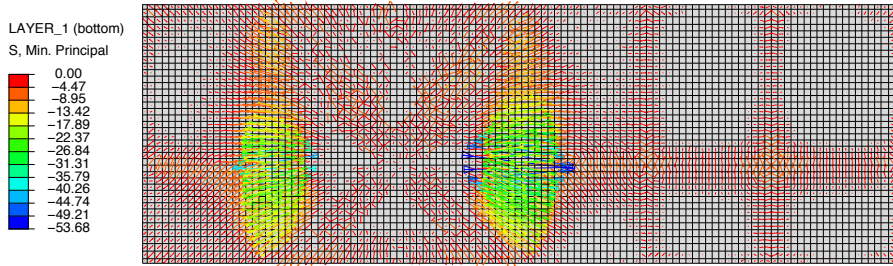
Figure A.3: Stresses in the slab - symmetric configuration - column B3 removal (units in MPa)



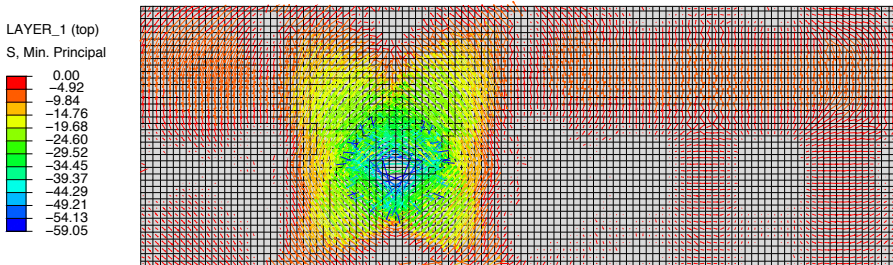
(a) Tensile stresses - bottom side



(b) Tensile stresses - top side



(c) Compressive stresses - bottom side



(d) Compressive stresses - top side

Figure A.4: Stresses in the slab - asymmetric configuration - column B3 removal (units in MPa)

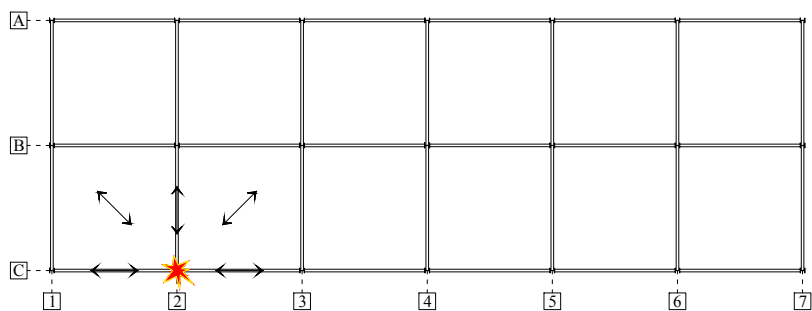


Figure A.5: Direction of the load redistribution - column C2 removal

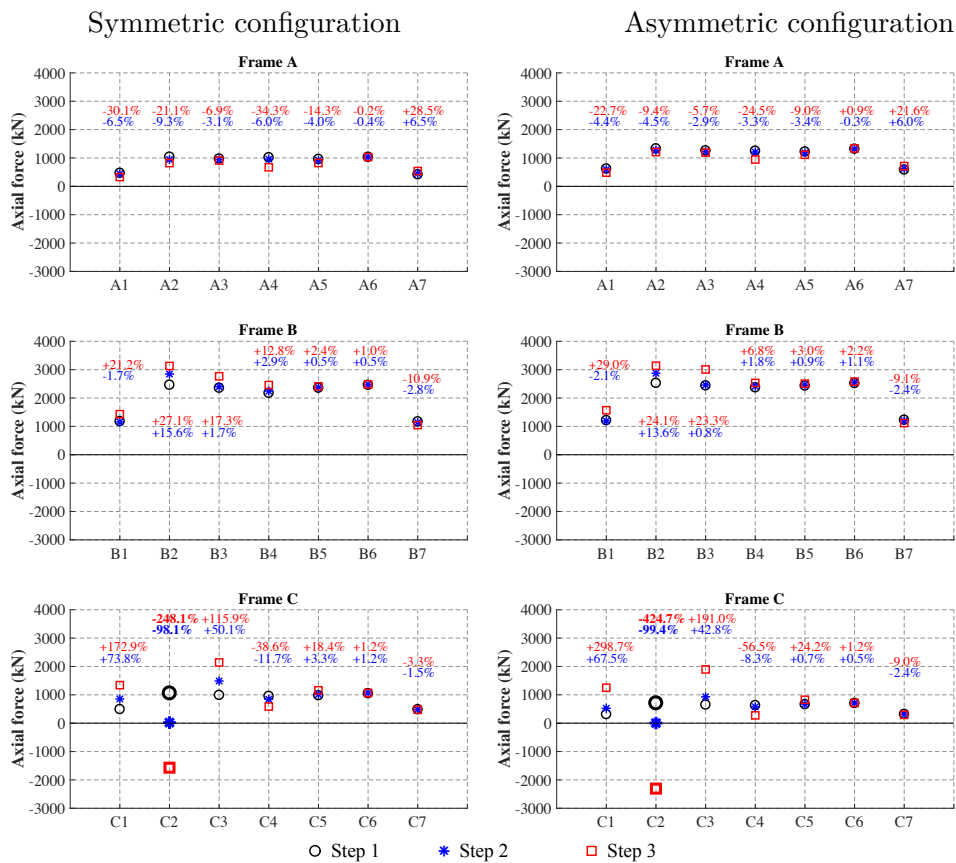


Figure A.6: Axial force in the columns - full-frame - floor 1 - column C2 removal

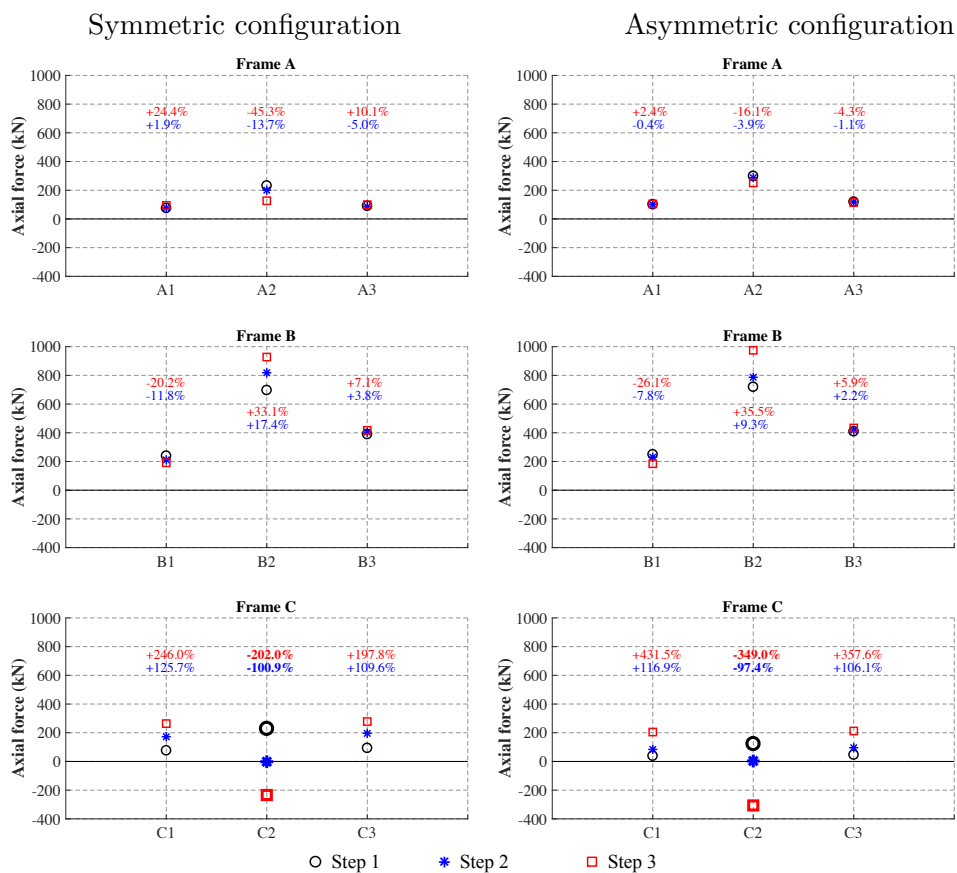
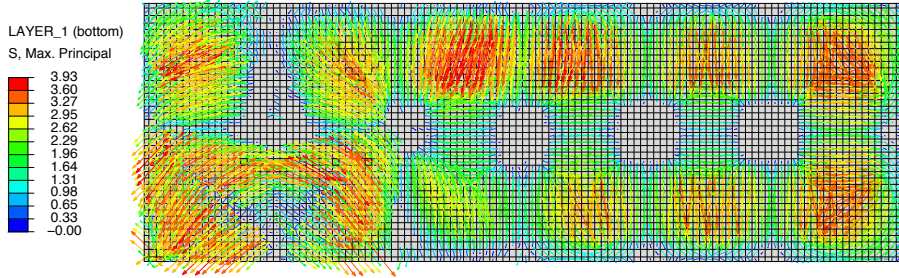
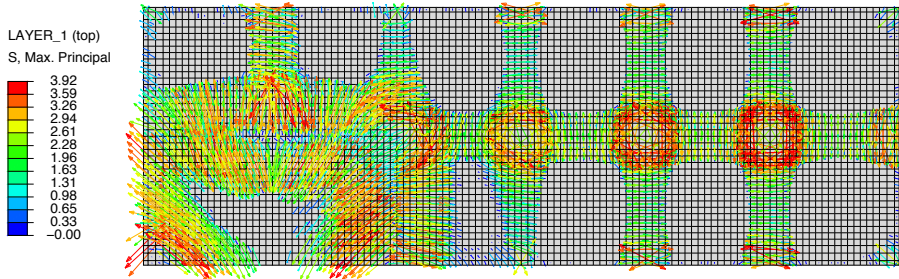


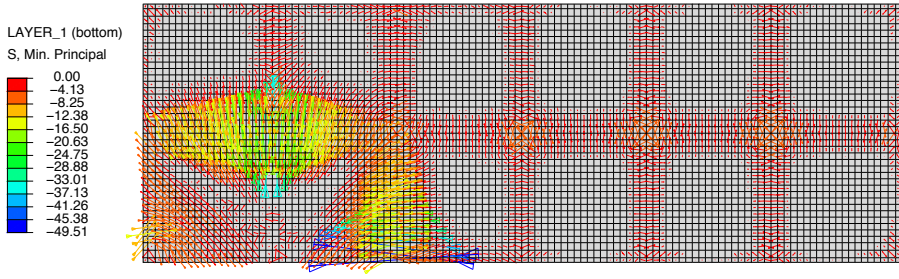
Figure A.7: Axial force in the columns - sub-frame - column C2 removal



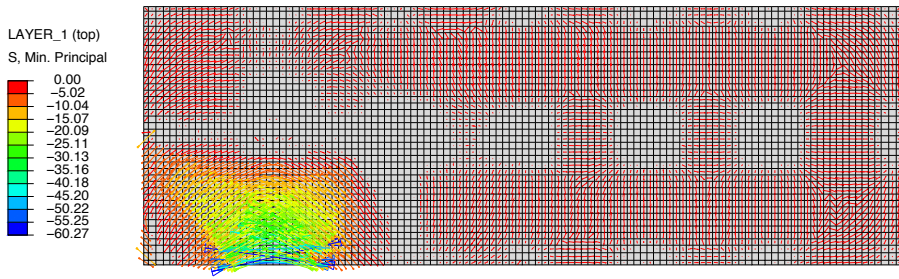
(a) Tensile stresses - bottom side



(b) Tensile stresses - top side

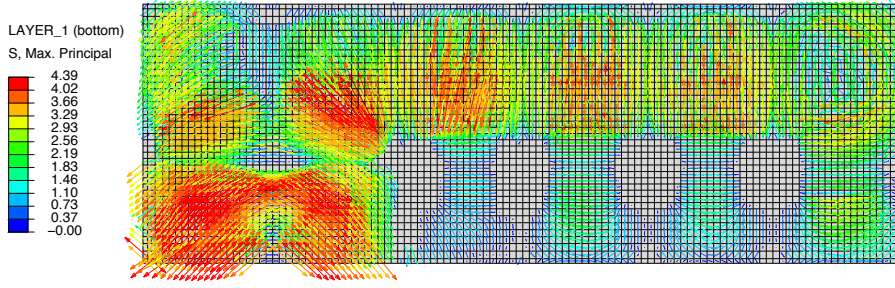


(c) Compressive stresses - bottom side

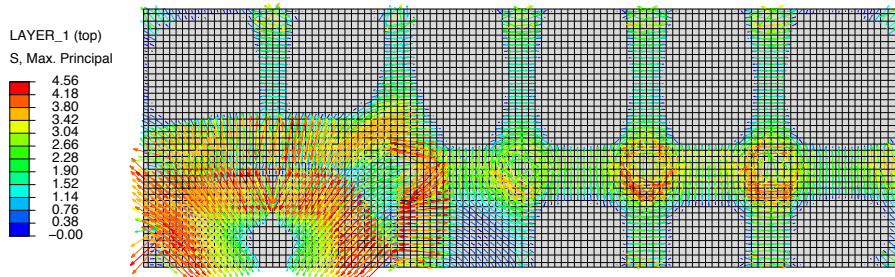


(d) Compressive stresses - top side

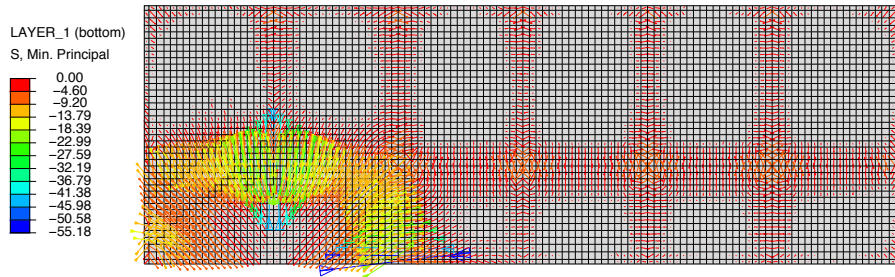
Figure A.8: Stresses in the slab - symmetric configuration - column C2 removal (units in MPa)



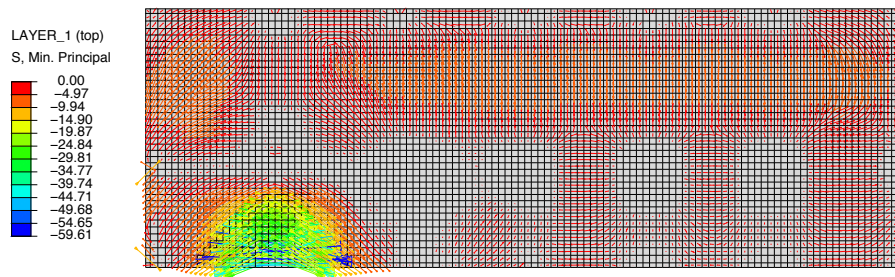
(a) Tensile stresses - bottom side



(b) Tensile stresses - top side



(c) Compressive stresses - bottom side



(d) Compressive stresses - top side

Figure A.9: Stresses in the slab - asymmetric configuration - column C2 removal (units in MPa)

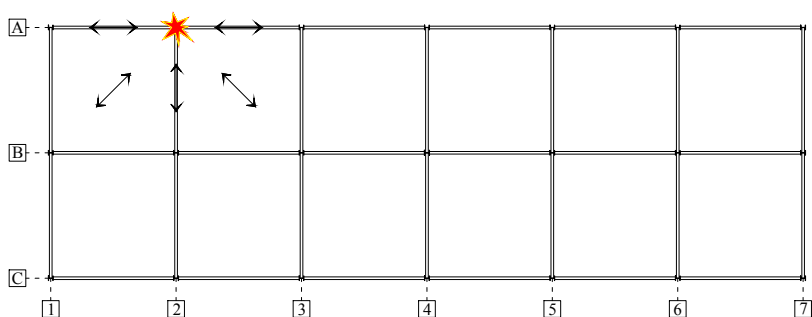


Figure A.10: Direction of the load redistribution - column A2 removal

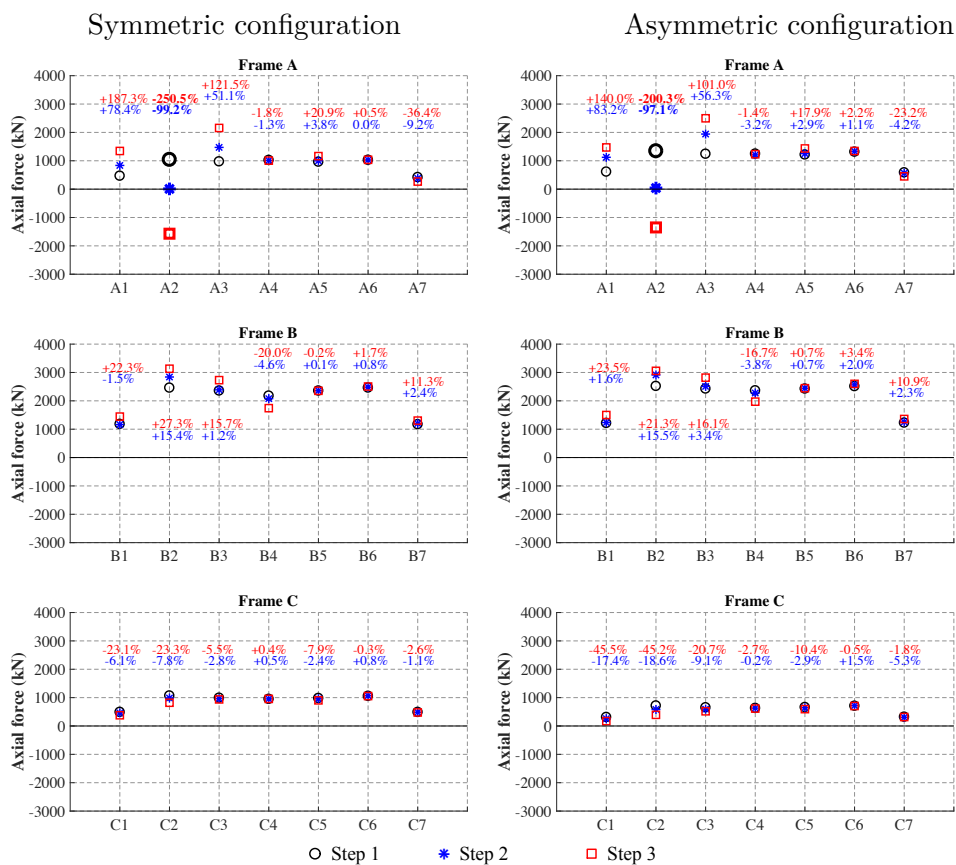


Figure A.11: Axial force in the columns - full-frame - floor 1 - column A2 removal

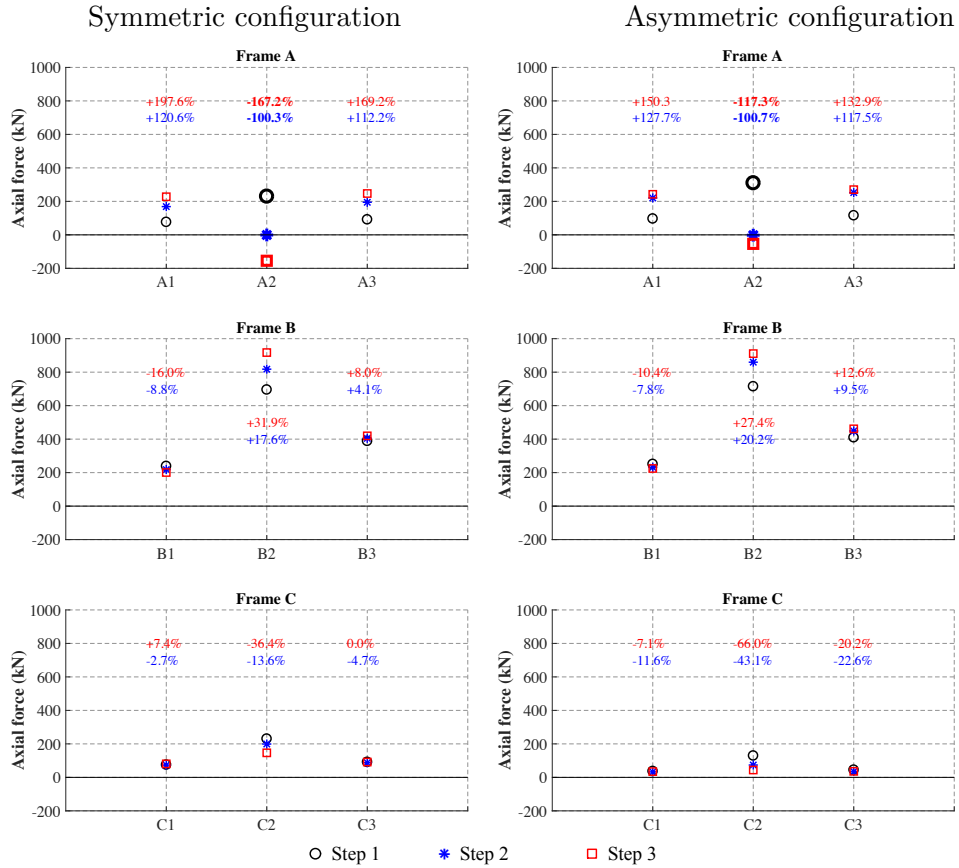
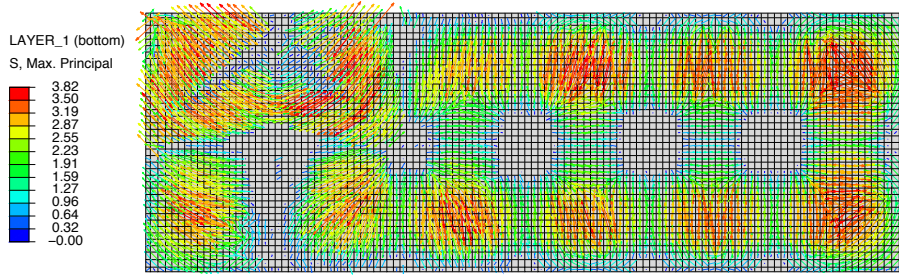
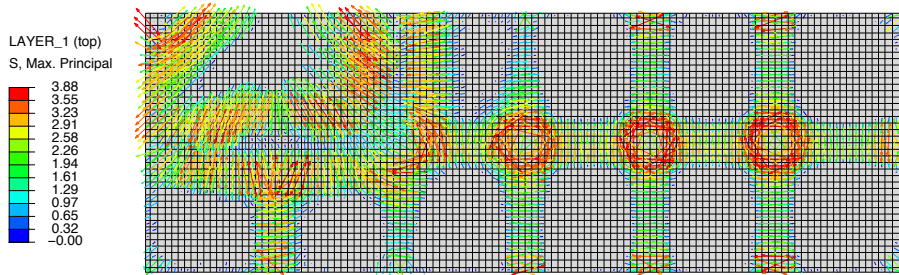


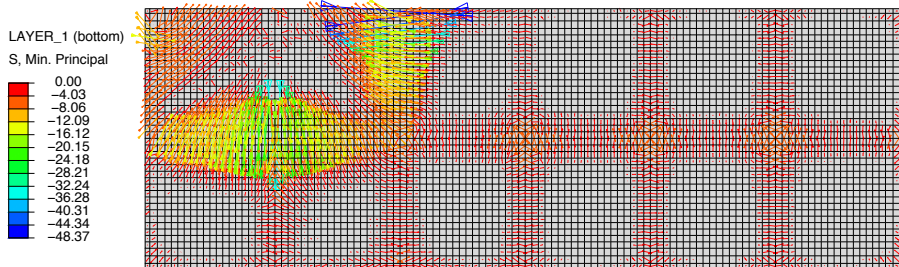
Figure A.12: Axial force in the columns - sub-frame - column A2 removal



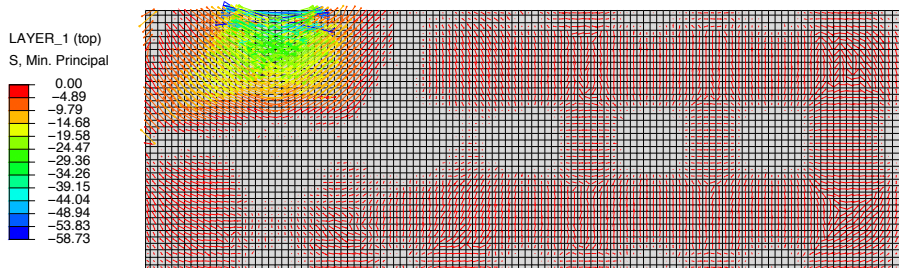
(a) Tensile stresses - bottom side



(b) Tensile stresses - top side

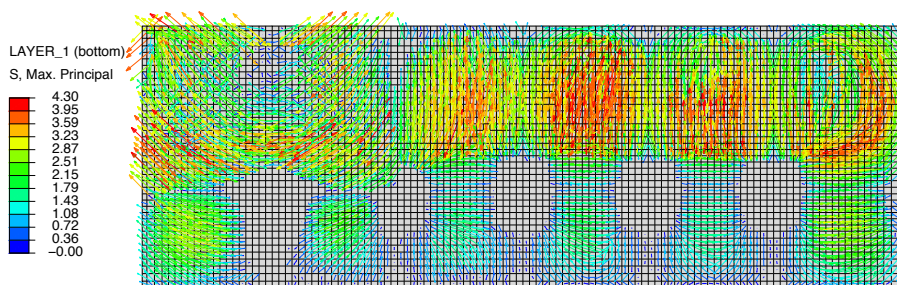


(c) Compressive stresses - bottom side

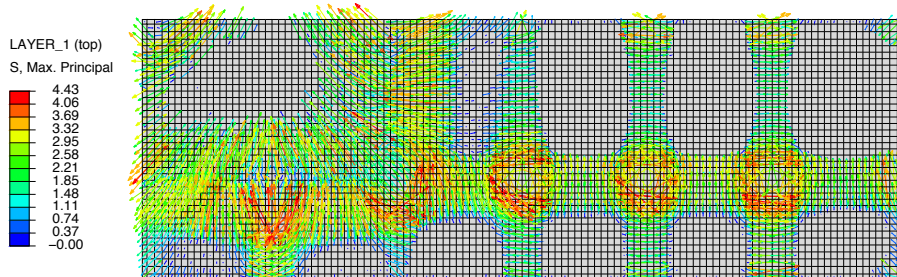


(d) Compressive stresses - top side

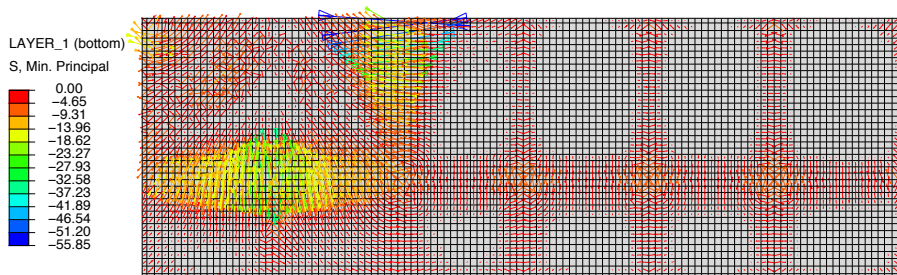
Figure A.13: Stresses in the slab - symmetric configuration - column A2 removal (units in MPa)



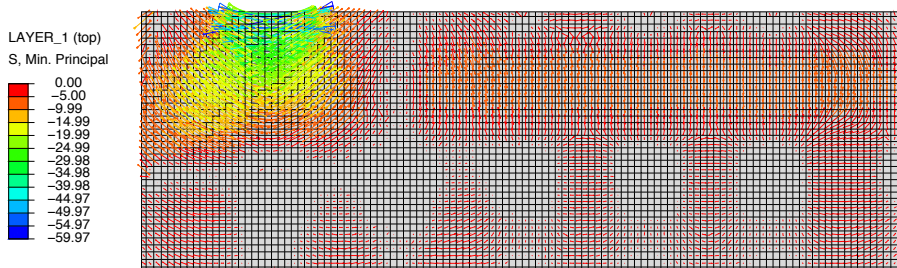
(a) Tensile stresses - bottom side



(b) Tensile stresses - top side



(c) Compressive stresses - bottom side



(d) Compressive stresses - top side

Figure A.14: Stresses in the slab - asymmetric configuration - column A2 removal (units in MPa)

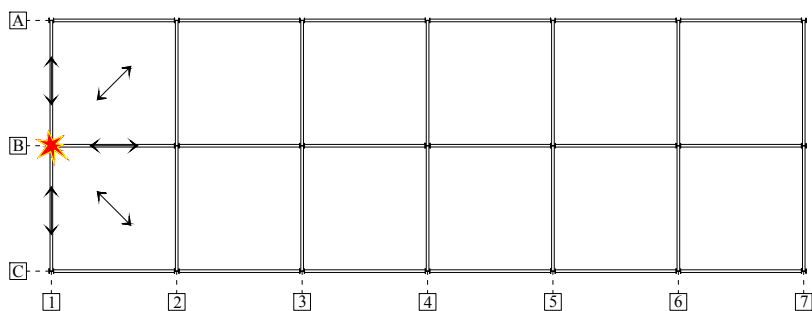


Figure A.15: Direction of the load redistribution - column B1 removal

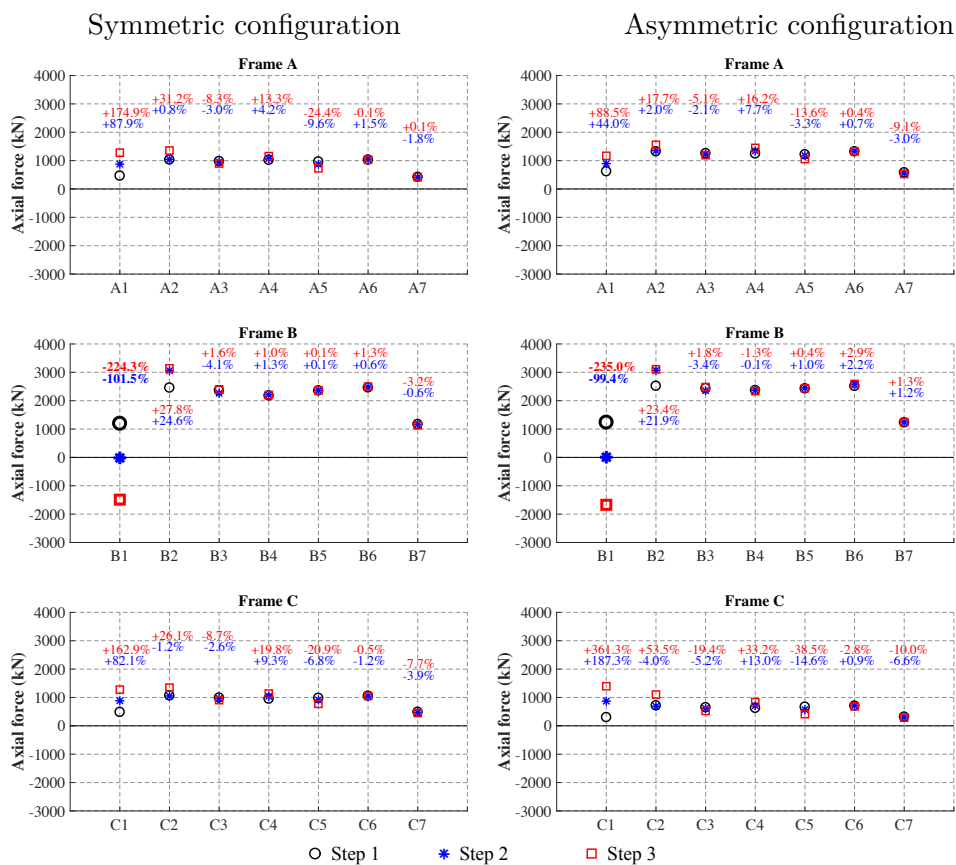


Figure A.16: Axial force in the columns - full-frame - floor 1 - column B1 removal

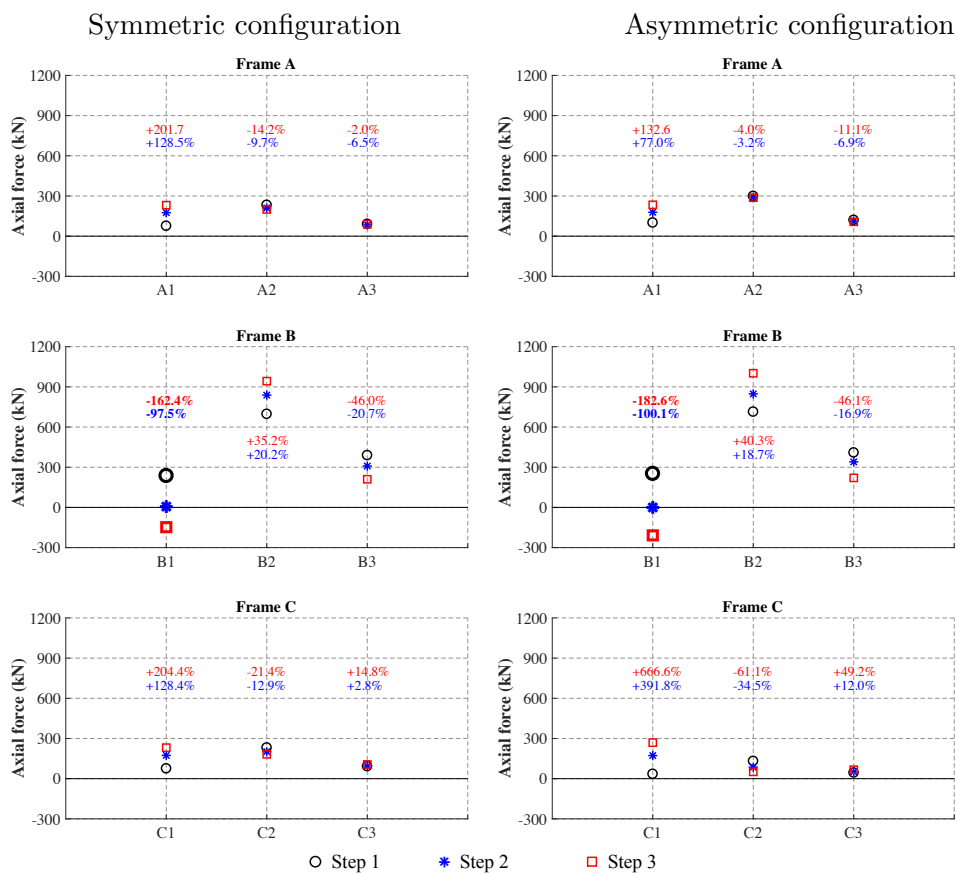
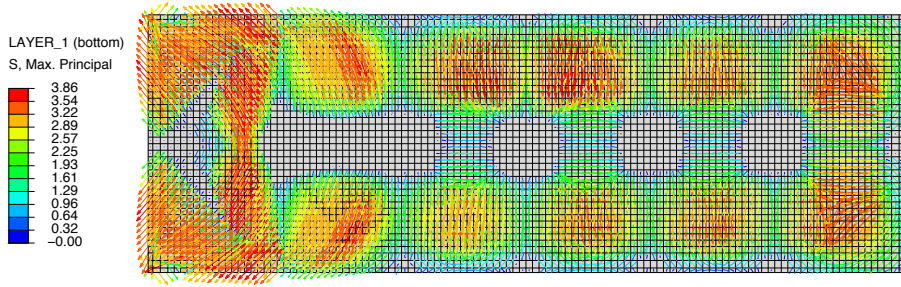
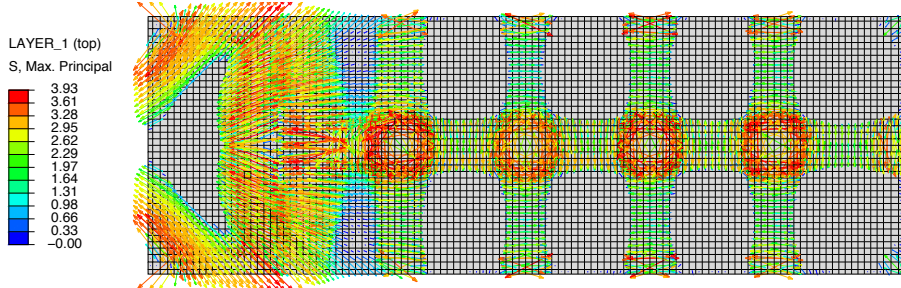


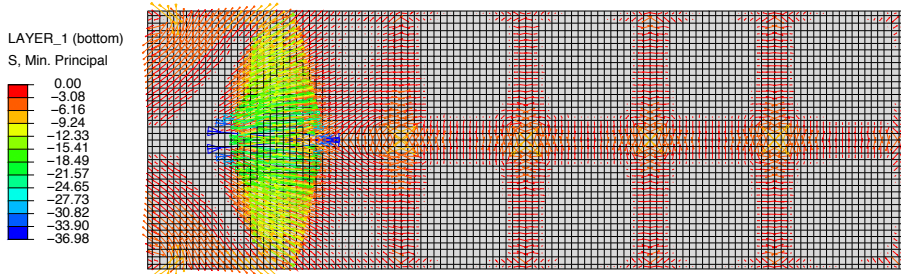
Figure A.17: Axial force in the columns - sub-frame - column B1 removal



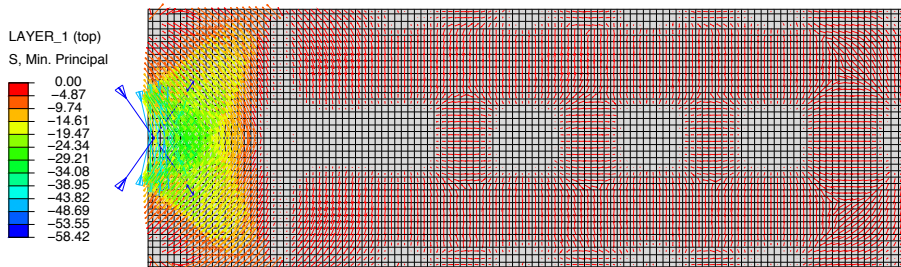
(a) *Tensile stresses - bottom side*



(b) *Tensile stresses - top side*

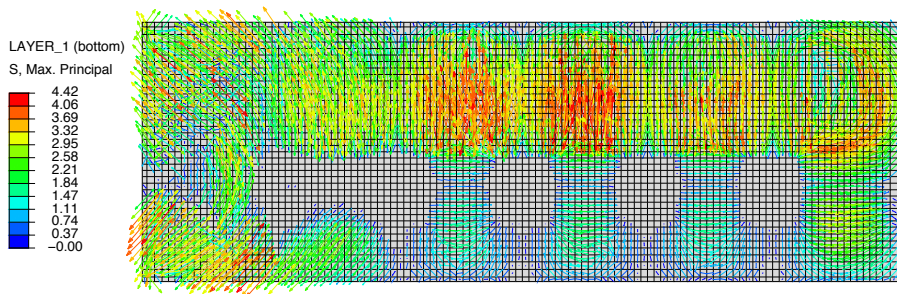


(c) *Compressive stresses - bottom side*

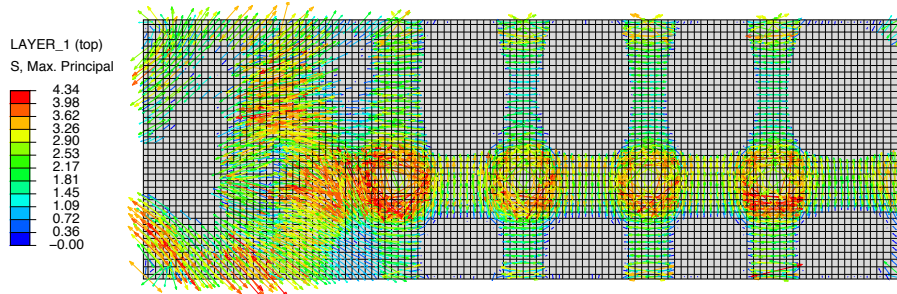


(d) *Compressive stresses - top side*

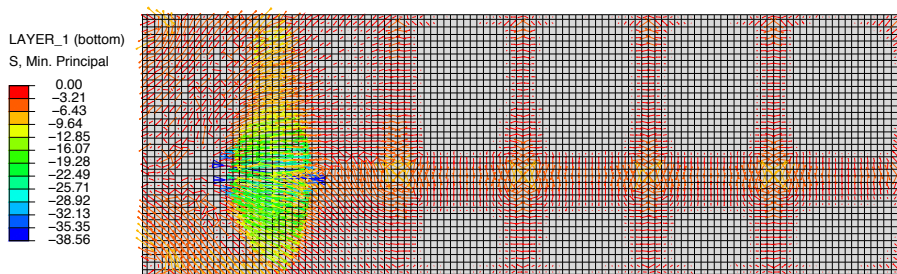
Figure A.18: *Stresses in the slab - symmetric configuration - column B1 removal (units in MPa)*



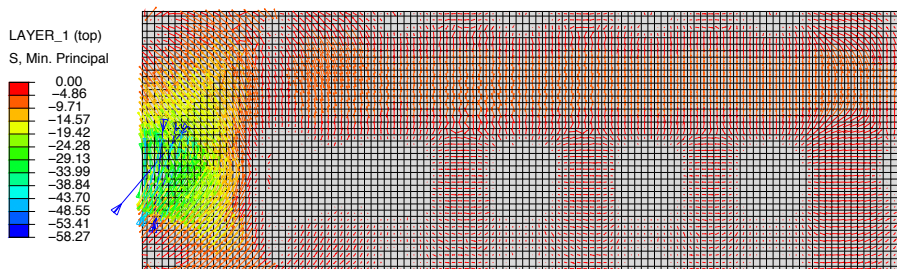
(a) *Tensile stresses - bottom side*



(b) *Tensile stresses - top side*



(c) *Compressive stresses - bottom side*



(d) *Compressive stresses - top side*

Figure A.19: *Stresses in the slab - asymmetric configuration - column B1 removal (units in MPa)*

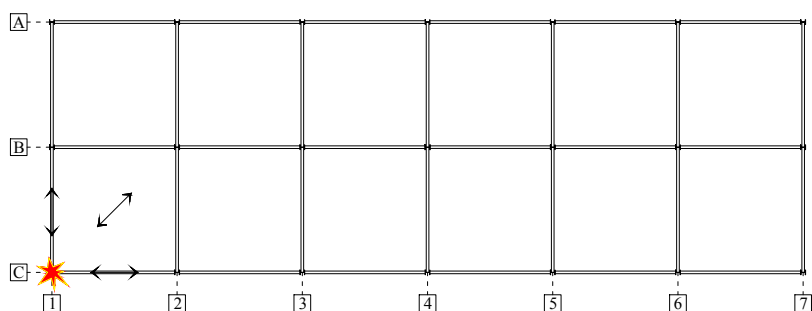


Figure A.20: Direction of the load redistribution - column C1 removal

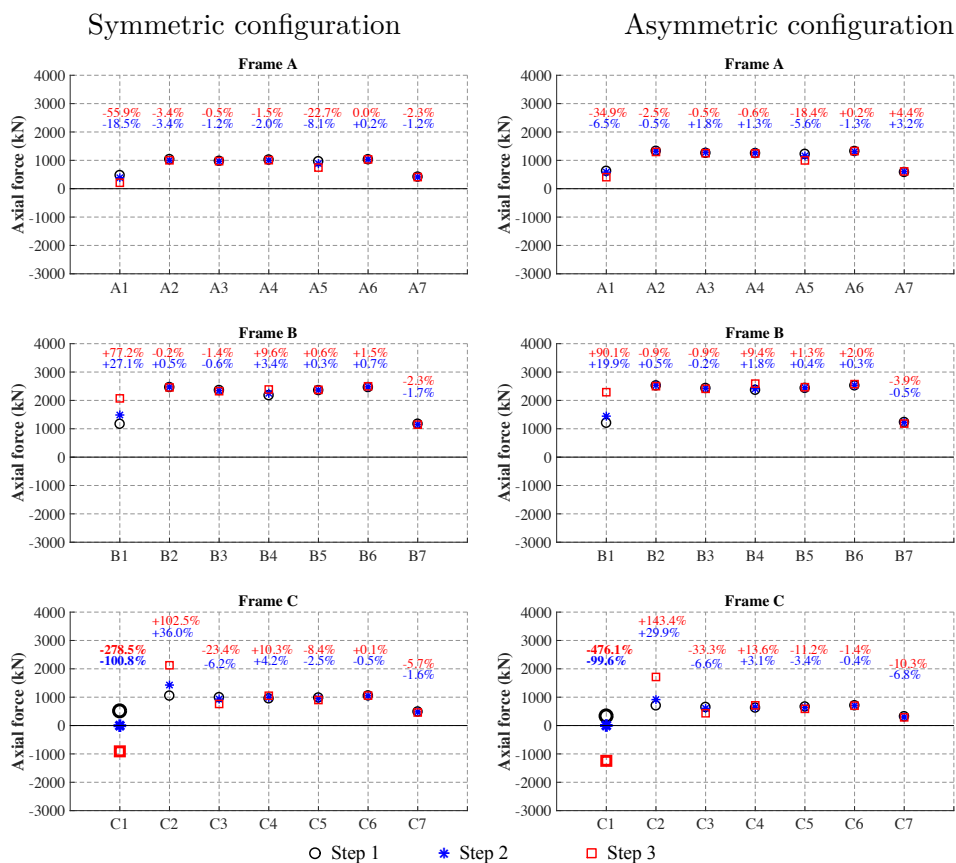


Figure A.21: Axial force in the columns - full-frame - floor 1 - column C1 removal

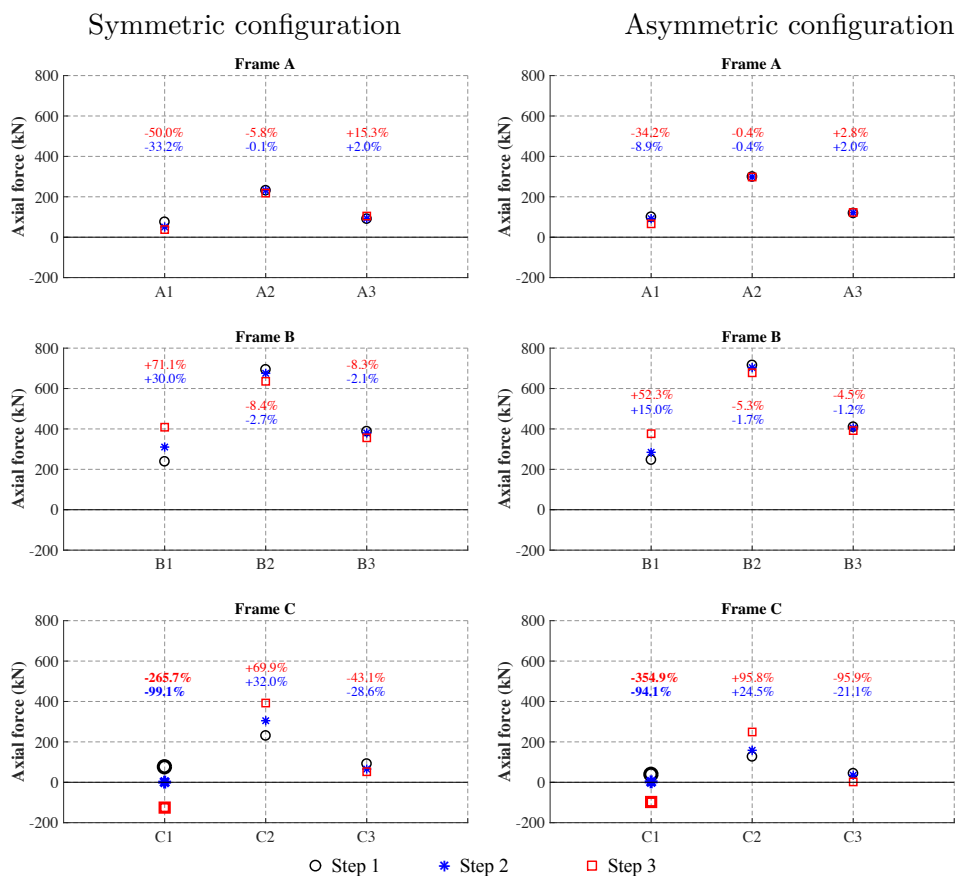
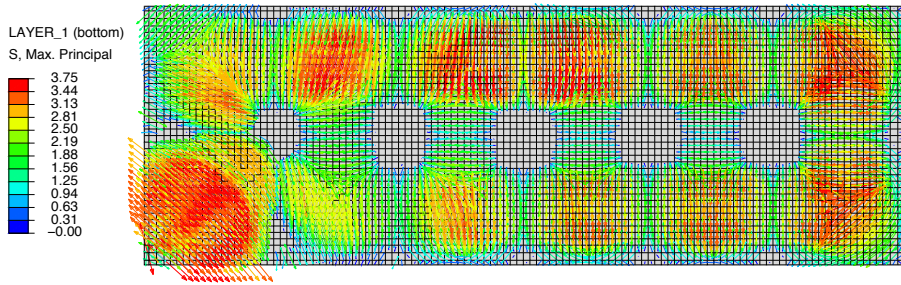
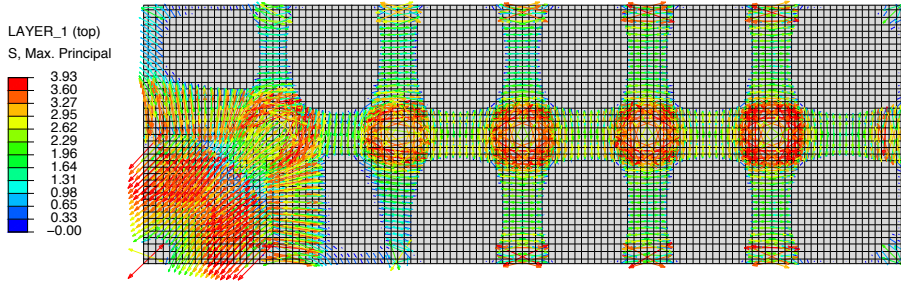


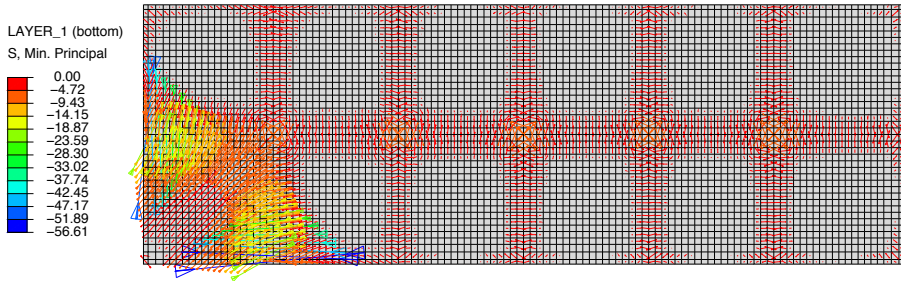
Figure A.22: Axial force in the columns - sub-frame - column C1 removal



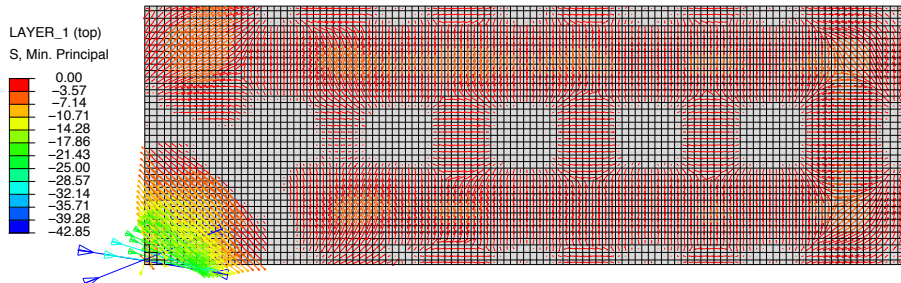
(a) Tensile stresses - bottom side



(b) Tensile stresses - top side

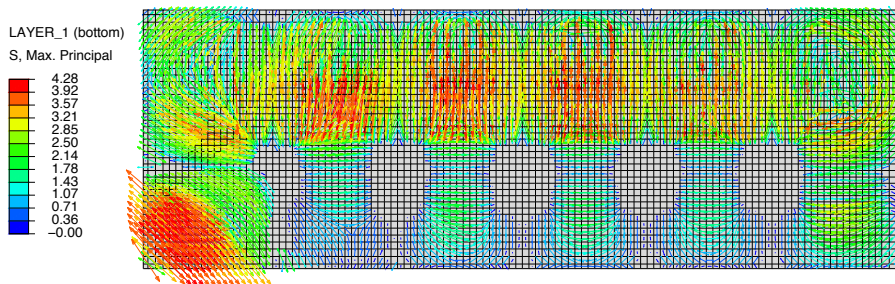


(c) Compressive stresses - bottom side

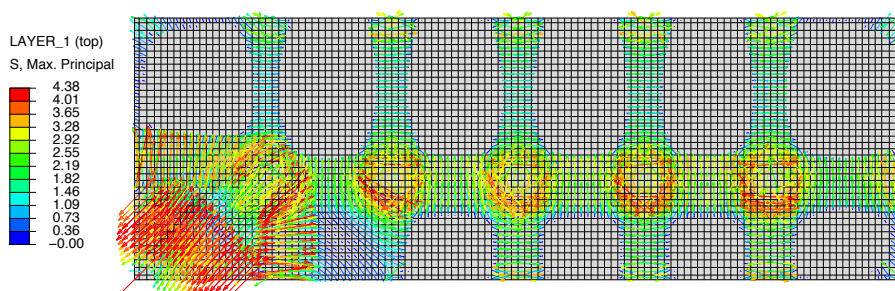


(d) Compressive stresses - top side

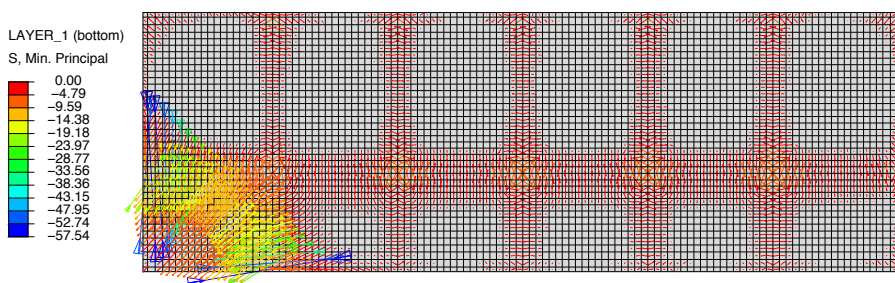
Figure A.23: Stresses in the slab - symmetric configuration - column C1 removal (units in MPa)



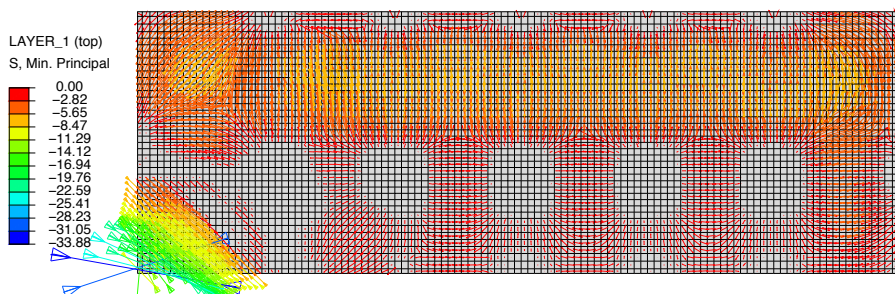
(a) Tensile stresses - bottom side



(b) Tensile stresses - top side



(c) Compressive stresses - bottom side



(d) Compressive stresses - top side

Figure A.24: Stresses in the slab - asymmetric configuration - column C1 removal (units in MPa)

Acknowledgements

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Accidental events, such as impact loading and explosions, are rare events with a very low probability of occurrence, but their effects often lead to very high human losses and economical consequences. Vulnerability of structures to the effects of local damages and its mitigation are issues widely discussed inside the scientific community. The structural property associated with such a vulnerability is named robustness. Depending on the type of the structural system and on the importance of consequences, specific design strategies can be adopted in order to ensure a robust structural response. Among them, the system redundancy, the joints and members ductility and the alternate load paths are the ones commonly adopted in case of multi-storey framed buildings. The present work focuses on the study of the behaviour of steel-concrete composite structures subjected to a column loss, and proposes a global overview to quantify the robustness of such systems subjected to this hazard scenario. The description of validated finite element models and of a new analytical tool to predict the response of flat concrete slabs subjected to large displacement are reported in this dissertation. Furthermore, important design hints for composite buildings are proposed.

Giacomo Roverso born in Arzignano, Vicenza, Italy in 1987. He received the Master degree in structural engineering from the University of Trento, Trento, Italy in 2015. He is currently pursuing a Ph.D. degree in modeling and simulation within the Doctoral Programme in Civil, Environmental and Mechanical Engineering at the University of Trento. His research focuses on steel and concrete composite structures subject to column loss.