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PROGRESSIVE COLLAPSE RETROFIT OF A STEEL-RC HOSPITAL BUILDING USING AN OUTRIGGER-BELT TRUSS SYSTEM

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Abstract

In recent years, there has been growing global concern regarding the potential for building structures to experience progressive collapse under abnormal loads. Numerous studies in the literature have focused on analyzing and designing methods to improve resistance against progressive collapse, and various technological retrofit solutions have been proposed. However, many of these studies rely on oversimplified models, limiting the applicability of their findings to practical scenarios. This paper introduces an alternative retrofit solution centered around an outrigger-belt truss system installed at the roof-top level of buildings. This system offers significant vertical stiffness and strength while requiring relatively small sections and minimal added mass. Additionally, it is characterized by its rapid, low-impact, and reversible implementation process, which allows the building to remain fully operational during the retrofitting process. This paper describes the progressive collapse analysis and retrofit of steel-reinforced concrete hospital buildings in Naples. A straightforward design procedure based on basic equilibrium principles is outlined. The effectiveness of this retrofit strategy is evaluated through non-linear static and dynamic analyses using real case studies, and its impact on existing structural members is discussed.

Keywords: Robustness, Progressive Collapse Retrofit, Nonlinear analysis.

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1 INTRODUCTION

The progressive collapse of hospital buildings, and vital infrastructures due to their strategic importance in emergencies, poses serious risks to human safety and structural integrity when subjected to abnormal loads, such as explosions, impacts, or other extreme events. Typically, buildings are designed to handle gravitational, seismic, and wind forces, yet they often lack provisions for unexpected events due to their unpredictable nature and intensity. This oversight can lead to catastrophic failure when primary load-bearing elements are compromised, triggering a progressive collapse. This vulnerability underscores the need for robust retrofit solutions that enhance the resilience of essential buildings to ensure operational continuity and life safety in the event of a structural compromise. Current retrofitting approaches to address progressive collapse are generally divided into two categories: local and global interventions. Local solutions, such as reinforcing beam-to-column connections or adding energy-dissipative devices to key structural components, offer targeted resilience but may not provide sufficient robustness across the entire structure. Global solutions, including vertical and horizontal bracing systems, cabling, and outrigger-belt trusses, provide more extensive protection by redistributing loads across the building, thus preventing progressive collapse in case of column or load-bearing element failure. However, such global retrofits often face challenges in terms of practical feasibility, economic cost, and significant construction impact, particularly in continuously operational facilities like hospitals. In this context, steel bracing, cabling, and roof mega trusses present a promising retrofit alternative, offering a way to enhance structural redundancy while maintaining the building's operational functionality. These structures can work alongside or independently of the existing load-bearing elements, distributing abnormal loads effectively and ensuring that critical areas of the building remain intact even in extreme conditions. This study introduces these novel retrofit methods to mitigate progressive collapse applied to real-world hospital buildings. Through nonlinear dynamic analyses, the performance of these systems in redistributing loads and reinforcing the structural integrity of hospitals under extreme conditions is explored. The findings demonstrate that such retrofits offer a feasible, low-impact, and sustainable solution for the progressive collapse retrofit of hospital facilities, balancing practical implementation with enhanced safety.

2 PROGRESSIVE COLLAPSE RETROFIT STRATEGIES

Generally, there are two main approaches to retrofitting existing buildings for progressive collapse resistance: local interventions and global interventions [1-9]. The first involves strengthening existing load paths through local interventions, such as reinforcing beams, columns, and beam-to-column connections, or adding energy dissipation devices in braces. A common technique for increasing the stiffness and strength of I-shaped beams in steel buildings, thereby enhancing resistance to progressive collapse, is to weld additional steel plates to their flanges. However, accessing the top flange for welding is often hindered by composite floor slabs, mechanical or electrical systems, horizontal bracings, or other obstacles. Consequently, flange plate welding or the addition of I-shaped sections is usually limited to the bottom flange of existing beams. Additionally, beams often lack sufficient flexural capacity because beam-column connections are typically simple (flexible). Limited tensile resistance further prevents the formation of a catenary mechanism, reducing the effectiveness of reinforcing beams with plates or I-sections. Generally, modifying beam-to-column and brace connections can significantly enhance the collapse resistance of steel frames. However, the insufficient tensile resistance of connections often limits the development of a full catenary mechanism. Strengthening connections also disrupt building operations during the retrofit process, a considerable issue for critical facilities like hospitals and emergency centers. A common technique for reinforced concrete (RC) buildings is reinforcing beams, columns, and beam-to-column joints by using Carbon Fiber Reinforced Polymer (CFRP) reinforcements or strips. The second approach focuses on creating alternative load paths through global interventions, such as cabling and bracing systems, top beam grids, or roof trusses. Another commonly used method to enhance collapse resistance in tall buildings is adding steel bracing or cabling systems across certain stories. However, retrofitting using steel bracing across selected stories may interfere with the building's function, making it economically challenging. An alternative is the installation of top beam grids that support columns from above in case of column loss. Often, though, existing buildings are not robust enough to withstand progressive collapse, requiring heavy steel cross-sections to increase vertical stiffness and strength through the flexural action of a top beam grid. This solution increases both the economic cost and seismic mass, which can elevate seismic responses. A more recent proposal involves top gravity truss systems, offering significant vertical stiffness and strength with relatively lighter sections and reduced mass. This technique is also quick to install, low-impact, reversible, and minimally affects the building's seismic response, allowing most areas to remain operational during retrofitting. Thus, a more cost-effective and efficient retrofit option for progressive collapse is to add a new structural system, such as a steel outrigger mega-truss on the building's top floor. The outrigger-belt truss system can be designed to carry all gravity loads following column loss without relying on the existing building’s load-bearing capacity. The sizing of mega truss members can be determined using basic equilibrium conditions under gravity loads.

3 BENCHMARK CASE STUDY: A STEEL-RC HOSPITAL BUILDING

3.1 Description

The case study focuses on the Santobono Hospital building in Naples. The original structure was a two-story reinforced concrete (RC) structure, which was then enclosed within a steel framework, thus creating two statically independent structures. The subject of this research work is the second building structure (Fig. 1). The mixed seismic-resistant system consists of a three-dimensional steel frame of columns and beams, vertical and horizontal steel braces, primarily of a pendular type, and a reinforced concrete (RC) stiffening core (Fig. 2). The primary beam-to-column connections are hinged, with continuous columns. Similarly, the connections between primary and secondary beams, as well as those between braces and columns, are also hinged. As a result, horizontal forces are absorbed by the vertical steel braces and the RC core. The floors consist of steel purlins and corrugated sheets, with live loads varying based on their intended use: 3.0 kPa for office spaces, 4.0 kPa for areas with moderate occupancy, and 5.0 kPa for zones expected to accommodate large crowds.

To assess the structural safety of the existing structure and determine the most appropriate level of safety against structural actions, it is essential to gather as much information as possible regarding the design history, geometry, and material properties of the load-bearing structures. In this specific case, knowledge of the building was obtained through historical research, studies of construction methods from the time of its construction, detailed surveys, instrumental investigations, and visual inspections. The characterization process was not limited to a simple interpretation and extrapolation of investigation results but was also the outcome of a more complex analysis of the building, including the identification of the principles and techniques used in its design and construction. The following steps were carried out: 1) Geometric and structural survey of both vertical and horizontal load-bearing elements, along with the identification of applied loads; 2) Analysis of construction details through pacometric investigations and direct inspections; 3) On-site testing and investigations, including visual inspections, core sampling, extraction of reinforcement bars, collection of steel samples from structural elements, removal of bolts, and indirect hardness testing. The level of knowledge of the building was achieved with a depth of analysis based on the accuracy of the survey operations, historical-documentary analysis, available documentation, and experimental investigations, reaching a knowledge Level classification of KL2 according to Italian and European standards [10-11], which corresponds to a Confidence Factor CF = 1.20. Regarding this achieved level of knowledge, the following aspects related to geometry, construction details, and materials are specified as follows. Geometry: Derived from the available documentation (where present) and a complete metric survey supplemented by various visual inspections. Construction details: Extensive investigations were conducted due to the lack of original project drawings. Material properties: Information on the mechanical characteristics of the materials was obtained by processing data from laboratory tests conducted on samples extracted on-site during the investigation campaign. The material strength values obtained were R_{cm}=26.8 MPa for concrete, f_{ym}=381.3 MPa for steel rebars, and f_{ym}=235 MPa for structural steel.

a) b)


Figure 1: External views: a) South Side; b) North Side.

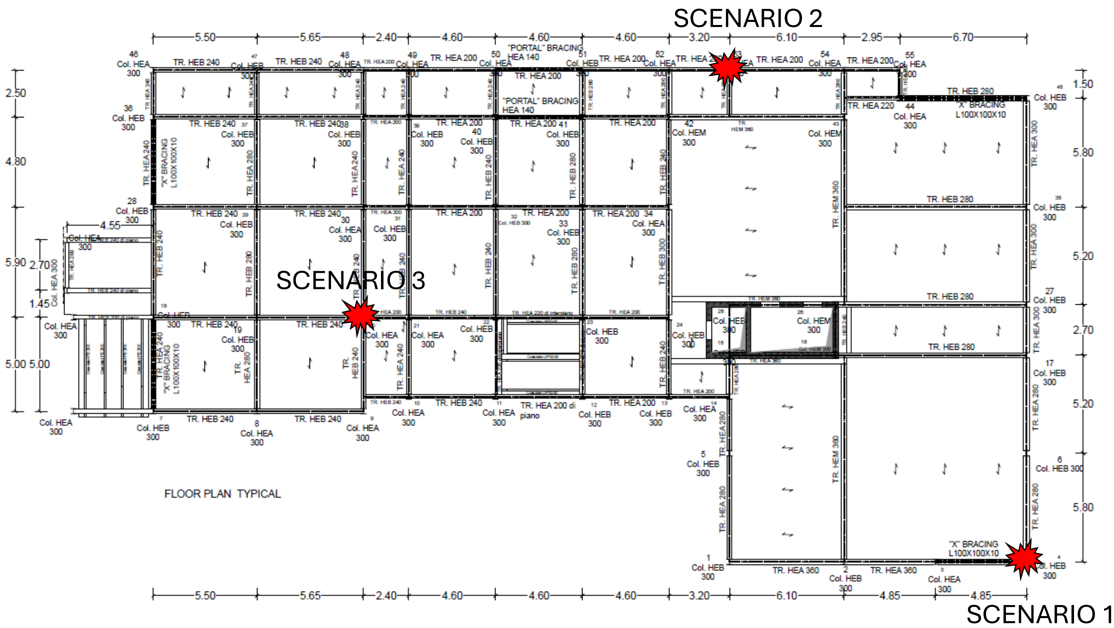
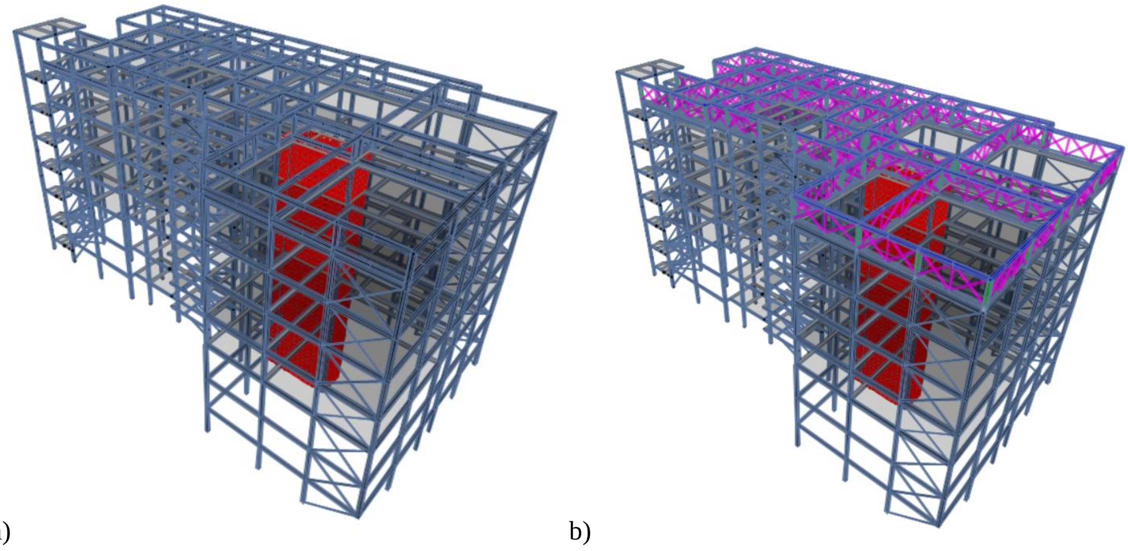


Figure 2: Plan View and Column Removal Scenarios.

3.2 Progressive Collapse Analysis

A detailed FEM model of the building is developed using SAP2000 [12] (Fig. 3a) incorporating both material and geometric nonlinearities. Steel columns and beams are represented as 2-node beam elements, while shear cores and walls are modeled with multi-layer shell elements. Nonlinear material behavior and large displacement effects alter the stiffness of structural members, which is accounted for along their length using a cubic deflection distribution for bending. To ensure numerical accuracy, beams are subdivided into twenty smaller frame elements. Initial geometric imperfections are disregarded, as they have minimal impact on progressive collapse scenarios in steel structures. The floor slabs are conservatively omitted from the model. The basement is assumed to have fixed supports, and the foundations are not explicitly included, following a conventional approach that disregards the impact of foundation settlement on structural behavior. The infill walls are made of double-layer masonry and, in several areas, contain large openings. They are considered only in terms of mass and, thus, their structural contribution (in terms of stiffness and strength) to the overall progressive collapse resistance of the building is neglected. For columns and beams, a concentrated hinge model is applied. According to GSA [13], their nonlinear modeling parameters and acceptance criteria are determined based on ASCE 41-13 [14]. In contrast, both GSA [13] and UFC [15] provide specific acceptance criteria for connections. In this case study, beam-to-column connections are considered simple (flexible) and are modeled as nominally pinned, meaning they are subjected only to axial forces. Their performance is evaluated, a posteriori, using Idea StatiCa [16]. The progressive collapse analysis follows the Alternate Path Method (APM) as specified in GSA [13] and UFC [15] guidelines. This approach assesses structural resilience by simulating the loss of critical columns. The study examines the structural response to the removal of a critical column, assessing the building's ability to withstand this extreme scenario. Three column removal cases are considered: (1) Corner column; 2) Side column; 3) Internal column (Fig. 2). The progressive collapse assessment is carried out using nonlinear static (pushdown) analysis. This static approach has a significant limitation: it does not capture key dynamic effects following sudden column removal, including transient dynamic response, and it does not account for dynamic amplification. To compensate for this limitation, current guidelines [13-15] introduce a Dynamic Increase Factor (*DIF*) applied to the spans of the removed column.



a) b)

Figure 3: Structural Model. a) As-built building; b) Retrofitted Building.

Specifically, the load combination DIF × LF × (1.2D + 0.5L) is used in these spans, while LF × (1.2D + 0.5L) is applied elsewhere, where LF is the load factor, D is the dead, and L is the live load. The load factor LF = 1 represents the application of the full gravity loads. The DIF to be used is a controversial issue in literature [17,18], However, the case study involves pinned frames, and, in this case, is appropriate to adopt the maximum value of 2.0 for DIF, that is the value recommended by earlier guidelines [19,20]. Fig. 4 shows the load factor vs vertical displacement (pushdown curve) in the location of the removed column (i.e., a corner column for Scenario 1, a side column for Scenario 2, an internal column for Scenario 3). On the pushdown curve are reported the points corresponding to the different limit states that occur: a) failure of a compressed brace; b) failure of a tensile brace connection; c) failure of a beam-to-column connection due to catenary action; d) failure of a compressed column. Fig. 5 shows the plastic hinge distribution corresponding to the last step of the pushdown analysis. For Scenario 1, the failure of compressed brace and tensile brace connections occurs respectively for LF=0.510 and LF=0.698. Then, the structure becomes unstable, so the remaining portion of the pushdown curve refers to a scenario where connection failure is prevented through an appropriate strengthening intervention. For scenarios 2 and 3, progressive collapse involves only pinned frames, with no contribution from braces. As a result, the pushdown curve lacks both the linear elastic and plastic phases, while its upward concavity signifies the activation of second-order stiffness. The tensile failure of the beam-to-column connection is evaluated retrospectively using Idea StatiCa software [16], occurring at LF = 0.55 for Scenario 2 and LF = 0.42 for Scenario 3. Consequently, in both cases, the remaining portion of the pushover curve develops only if connection failure is prevented. Ultimately, in all the column removal scenarios analyzed, the load factor remains below 1, indicating that the structure requires retrofitting to prevent progressive collapse.

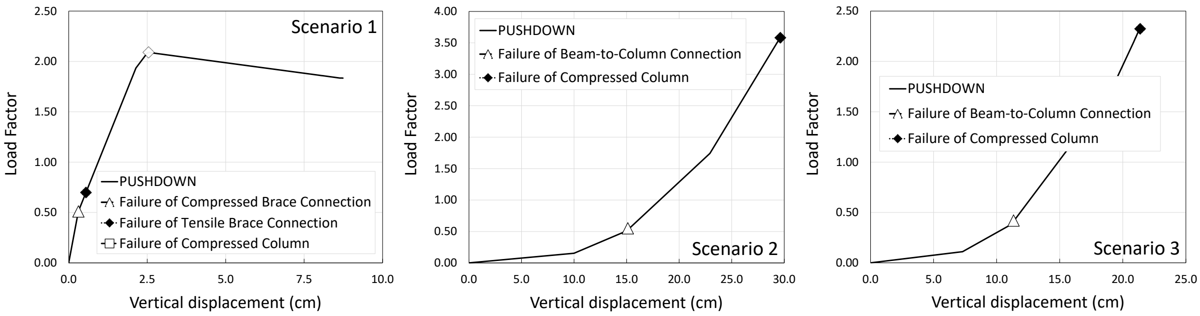
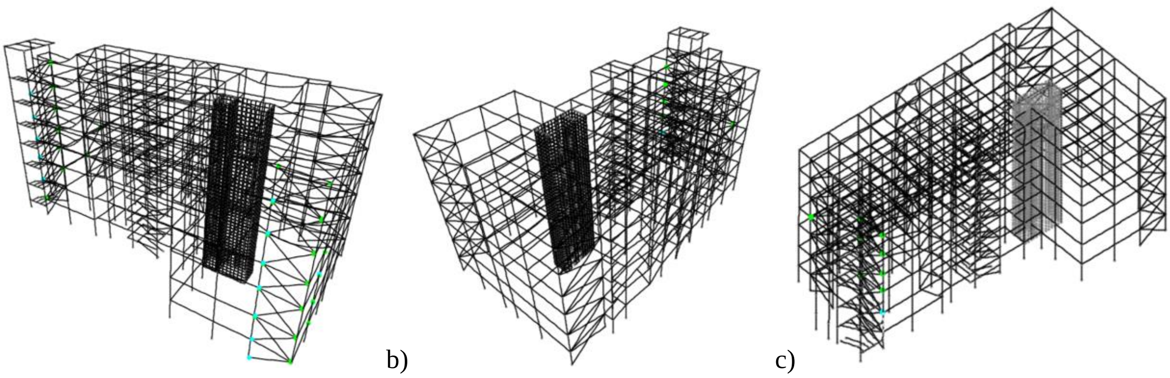


Figure 4: Pushdown Curves (Load factor vs. Vertical displacement) under various column removal scenarios.



a) b) c)

Figure 5: Plastic hinge distribution at the last step of the pushdown analysis.

4 PROGRESSIVE COLLAPSE RETROFIT USING AN OUTRIGGER-BELT TRUSS SYSTEM

4.1 Design Strategy

Retrofitting an existing building to enhance its resistance against progressive collapse can generally follow two approaches. The first involves reinforcing the existing load paths, while the second focuses on creating alternative load paths. The first approach typically includes localized interventions, such as strengthening beams, columns, or beam-to-column connections, or integrating energy dissipation devices within the bracing system. Conversely, the second approach relies on global interventions, such as adding cables, bracing systems, top beam grids, or roof trusses to redistribute loads effectively. In the case study, the beam-to-column connections are nominally pinned, leading to minimal flexural capacity in the beams after column removal. Additionally, the poor tensile axial strength of the beam-to-column connections restricts the formation of a catenary mechanism, which is crucial for redistributing loads in a progressive collapse scenario. Strengthening all beam-to-column connections is not a feasible option due to the high cost and disruption to building operations, which is particularly critical for hospital facilities. Similarly, retrofitting with steel bracings across specific stories is not a practical solution. As an alternative, a retrofit strategy based on an outrigger-belt truss system has been proposed in recent studies [5,7,9,21]. This system, installed at the roof level and connected to the column ends, introduces alternative load paths, enhancing both structural redundancy and robustness (Fig. 3b). The design of the top gravity truss system is carried out without considering the existing building's load-bearing capacity—a decision justified by the weak progressive collapse resistance of the original structure. Since the beam-to-column connections are flexible (nominally pinned), their tensile axial resistance is low, limiting the catenary effect and, consequently, the building's ability to resist progressive collapse (LF = 0.51 for Scenario 1, LF = 0.55 for Scenario 2, and LF = 0.42 for Scenario 3). Thus, the only viable solution to enhance collapse resistance without strengthening all connections is to limit the increase in vertical displacement following column removal. To achieve this, the outrigger-belt truss system is designed to ensure that the structure operates within the small displacement regime, where the flexural stiffness of the original structure remains negligible due to the pinned connections. Consequently, the load-carrying contribution of the existing building is disregarded without compromising structural integrity. The truss system is designed to support all gravity loads after column removal, assuming that the original structure provides no additional support. For preliminary member sizing, equilibrium conditions are used to estimate axial forces. The system is constructed with S235 structural steel and full-penetration welded joints, using the following cross-sections: HEB300 for vertical and horizontal webs, C219.1x12.5 for diagonal struts and top chords. A detailed finite element model of the outrigger-belt truss system is developed using SAP2000 [12], where the truss is modeled as a set of two-node finite elements with three degrees of freedom per node (Fig. 3b).

4.2 Effectiveness of the retrofit solution

The effectiveness of the proposed retrofit solution is evaluated using both nonlinear static (pushdown) and nonlinear dynamic analysis. To achieve this, axial plastic hinges are introduced at the ends of the truss beams, with parameters defined according to ASCE 41-13 [14]. In the pushdown analysis, gravity loads are first applied to the undamaged structure, followed by the removal of a critical column. The gravity loads in the affected bays are amplified, and an amplification factor of 2.0 was deemed conservative, as the stiffness of the roof truss limited vertical deflections thus preventing significant catenary effects. Fig.6 presents the load-displacement curves of the retrofitted structure for the three different column removal scenarios, while Fig. 7 illustrates the corresponding plastic hinge distribution at the final step of the pushdown analysis. The results confirm the effectiveness of the retrofit strategy. As shown in Fig. 6, the load factor exceeds one in all scenarios, demonstrating improved structural resilience. The outrigger-belt truss system effectively redistributes gravity loads following column removal, with the columns above the removed column functioning primarily as tie rods. Ultimately, failure occurs in a compressed column.

Finally, a nonlinear dynamic analysis is performed using the load combination (1.2D + 0.5L), and incorporating a bilinear kinematic hysteretic model for steel and a proportional damping model based on the first and second vertical mode shapes of the damaged structure. A conservative damping ratio of 2% is considered, particularly for bolted connections. Similar to the static analysis, ASCE 41-13 [15] acceptance criteria are applied to beams and columns, while GSA [13] guidelines are used for connections. The dynamic response is assessed by measuring the vertical displacement at the removed column location. Fig. 8 illustrates the time history of vertical displacement at the removed column location. Results indicate a significant reduction in dynamic response after retrofitting: Maximum vertical displacement in the retrofitted structure is 4.96 mm for Scenario 1, 9.77 mm for Scenario 2, and 14.1 mm for Scenario 3. All structural members remained within their elastic limits, except for the compressed braces in Scenario 1. The acceptance criteria for beams, columns, and connections were never exceeded during dynamic analysis. This confirms the effectiveness of the retrofit strategy in significantly improving the progressive collapse resistance of the building.

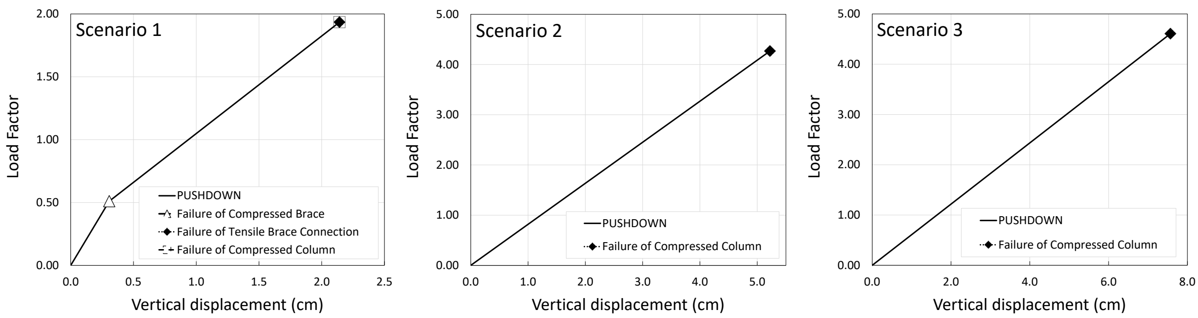


Figure 6: Pushdown Curve (Load factor vs. Vertical displacement).

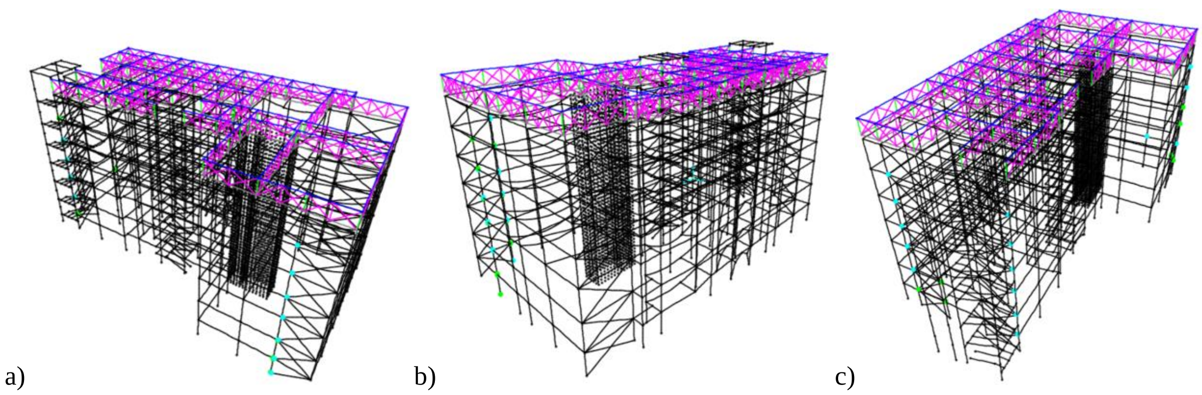


Figure 7: Deformed shape and plastic hinges at the last step of the pushdown analysis.

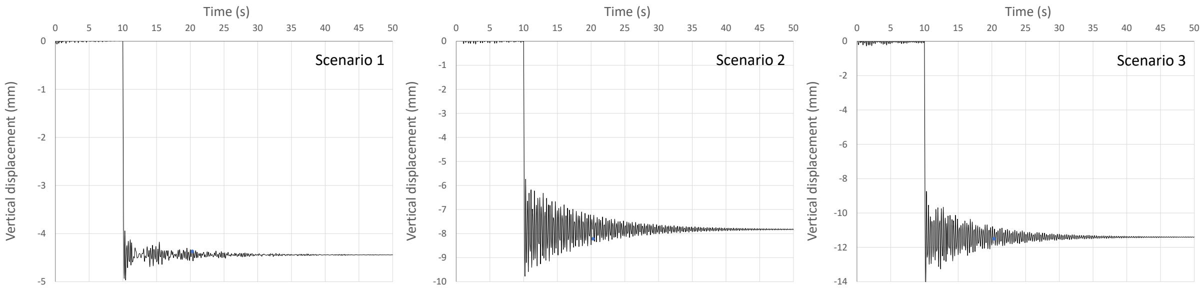


Figure 8: Displacement time-histories

5 CONCLUSIONS

This study presents a retrofit strategy using an outrigger-belt truss system to enhance the progressive collapse resistance of a steel-reinforced concrete hospital building. The structural performance was assessed through nonlinear static (pushdown) and nonlinear dynamic analyses, demonstrating the effectiveness of the proposed solution. Key findings from the analyses are summarized below. In the as-built structure, the load factor remained below 1 for all column removal scenarios, indicating a high susceptibility to progressive collapse. The outrigger-belt truss system effectively redistributed gravity loads following column removal, significantly improving the load-bearing capacity and reducing vertical displacements. The retrofitted structure exhibited a load factor greater than 1 in all scenarios, confirming a substantial increase in robustness and structural redundancy. The dynamic analysis showed that the vertical displacements are very limited, indicating a marked improvement in structural stability. All structural members remained within their elastic limits, except for localized compression failures in braces, which did not compromise overall stability.

The outrigger-belt truss retrofit strategy offers a low-impact, reversible, and cost-effective solution that maintains the building's full functionality during implementation. This approach ensures that essential facilities, such as hospitals, remain operational while significantly enhancing their resilience against progressive collapse. Future research may focus on expanding this methodology to other building typologies and refining design procedures to further optimize structural performance under extreme loading conditions.

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