Roof-level retrofitting beams to prevent progressive collapse: a study on their effectiveness in a steel building

Vigas de refuerzo a nivel de la cubierta para evitar el colapso progresivo: estudio de su eficacia en un edificio de acero

Marcos Arias-Gracey[a], Brais Barros[b], Lisbel Rueda-García[c], Manuel Buitrago[d] y José M. Adam[e]

[a] Civil Engineer. Universitat Politècnica de València. Researcher. PhD Student. marigra2@upv.es

[b] PhD Industrial Engineer. Universitat Politècnica de València. Postdoctoral Researcher. bbargon1@upv.es

[c] PhD Civil Engineer. Universitat Politècnica de València. Postdoctoral Researcher. lisruega@upv.es

[d] PhD Civil Engineer. Universitat Politècnica de València. Postdoctoral Researcher. mabuimo1@upv.es

[e] PhD Civil Engineer. Universitat Politècnica de València. University Professor. joadmar@upv.es

RESUMEN

La creciente frecuencia de eventos extremos resalta la necesidad de diseños estructurales robustos, especialmente en edificios donde los daños locales pueden conducir al colapso. Este estudio propone un método práctico de refuerzo utilizando vigas rígidas en el nivel de cubierta para crear caminos alternativos de carga en caso de fallos de columnas. Se realizó un análisis por elementos finitos en un edificio de acero representativo, evaluando múltiples escenarios de fallo de columnas y configuraciones de refuerzo. Los resultados demuestran la eficacia de diferentes configuraciones de vigas para prevenir el colapso progresivo, ofreciendo una solución práctica para mejorar la robustez estructural.

ABSTRACT

The increasing frequency of extreme events highlights the need for robust structural designs, especially in buildings where local damage can lead to collapse. While retrofitting solutions exist, many are inefficient or impractical. This study proposes a practical retrofitting method using stiffening beams at the roof level to create alternative load paths in the event of column failures. A finite element analysis was conducted on a representative steel building, assessing multiple column failure scenarios and retrofitting configurations. The results demonstrate the effectiveness of the optimal beam configuration in preventing progressive collapse, offering a practical solution for enhancing structural robustness.

PALABRAS CLAVE: robustez, colapso progresivo, edificio de acero, viga de refuerzo, ALPs, MEF

KEYWORDS: robustness, progressive collapse, steel building, retrofitting beam, ALPs, FEM

1. Introduction

Throughout history, localised damage to structures has frequently triggered severe failures, sometimes resulting in partial or total collapse. This risk is heightened by the growing frequency of extreme events, such as extreme meteorological events, vehicle impacts, terrorist attacks, or blasts, emphasising the urgent need to ensure the robustness of buildings, particularly critical structures where failure could result in significant loss of life. Although many countries have developed specific standards for structural robustness in recent decades [1,2], the majority of existing buildings were designed before these standards were introduced and, with few exceptions, do not incorporate design provisions to ensure structural robustness. In fact, the risk of progressive collapse is often particularly high in steel structures due to their generally low level of redundancy [3].

Fortunately, significant progress has been made in addressing progressive collapse over the last few decades, and many solutions have been developed to enhance the robustness of existing buildings. However, many of these solutions can be prohibitively expensive, invasive, complex to implement, involve lengthy operational interruptions, or are of local nature and therefore generally ineffective. This makes the solution intrusive and operationally costly. For example, Papavasileiou and Pnevmatikos [4] proposed to use steel cables as a structural retrofit for seismically designed steel-concrete composite buildings. While this retrofit can effectively increment the building's redundancy, it also severely changes its dynamic behaviour, which could negatively affect its seismic performance. Several studies [5–7] proposed local interventions such as increasing the strength and stiffness of beams or beam-to-column connections to enhance catenary action. However, this approach is not always effective, particularly in cases where the columns are the weakest elements, meaning improvements in beam behaviour would not significantly impact the overall structural performance.

A promising solution to enhance a structure's redundancy and resistance to progressive collapse involves adding stiffening beams on the roof. In the event of local failure of one or more columns on the lower floors, this global solution can enhance the existing load paths or generate an alternative one to prevent the collapse of the structure (Figure 1). Considering the retrofitting techniques mentioned before, this solution is far less intrusive and the probability of interference with the building's regular operations is significantly lower. In addition, as the retrofit is at the roof level, it is less exposed to external factors such as vehicle impact, flooding, etc. This solution has been explored by Ferraioli et al. [3], who proposed a retrofit for an existing hospital building using an outrigger-belt truss system connected at the rooftop level to the ends of each column. However, their approach did not include an optimization process for the reinforcement layout design. Additionally, Freddi et al. [8] investigated the effectiveness of the solution and its resisting mechanisms by modelling a single plane frame of a 9-story Moment Resisting Frame steel building and adding a steel truss system at the rooftop level.

In the present work, a numerical study is carried out using finite element models to evaluate the effectiveness of the proposed retrofit system in a steel frame building. For this purpose, a steel building representative of Spanish construction is designed and modelled using the corresponding standards; thirteen possible failure scenarios are evaluated by simulating the collapse of different columns, and four different stiffening beam configurations that prevent the progressive collapse of the structure are studied. The optimal retrofit configuration is obtained based on key performance indicators in the structure. The results obtained show the effectiveness of this technique in improving the robustness of the steel building structure.

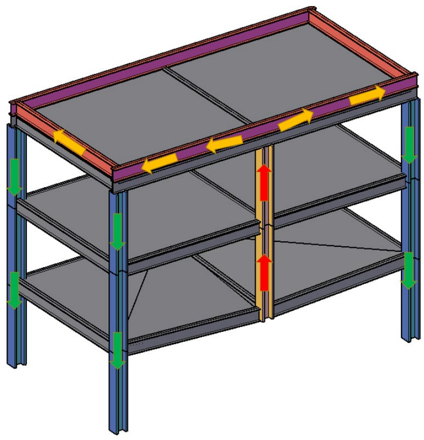

Figure 1. Conceptual diagram of the studied retrofitting technique against collapse.

2. Case study

In order to prove the effectiveness of the proposed retrofit, a 3-story steel building has been designed employing CYPECAD software [9]. The structure features seven 6-meter bays in one direction and three bays of the same size in the other (see Figure 2). Additionally, the story height is 4 meters, resulting in a total building height of 12 meters. The building, representative of typical Spanish steel construction, was designed in accordance with Eurocode regulations [10]. Coastal wind loads were considered; however, the design did not include seismic loads. For the design and modelling, steel grade S355 was used for beams, columns and the composite steel deck's steel sheet, with a Young's modulus of 210 GPa, a Poisson's ratio of 0.3, a yield stress of 355 MPa, and an ultimate stress of 490 MPa. C25/30 concrete was employed for the slabs and B500 steel bars for the slabs' negative reinforcement. As for the profiles used, standardised profiles were chosen in accordance with Eurocode guidelines [10], with IPE profiles used for the beams, while HEM profiles were employed for the columns. The column profiles were arranged so that the beams in the longer direction of the building would connect to the columns' flanges. This way, the beam-to-column connections along the longer direction were considered as full-strength rigid connections. In the shorter direction, the beams are welded to the column webs, which results in limited flexural strength. For the slabs, composite steel deck flooring was used, which, attached to the supporting beams via shear studs, was assumed to exhibit composite behaviour. The composite steel decks consist of a trapezoidal steel sheet, 0.8 mm thick and 60 mm tall, which acts as formwork during the construction stage and replaces the positive reinforcement in a regular concrete slab, topped with a 100 mm thick layer of concrete and negative reinforcement positioned 20 mm from the top of the slab.

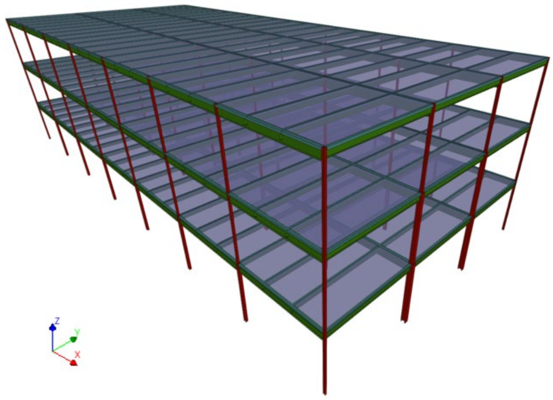

Figure 2. 3D view of the structure in CYPECAD software.

The designed building was subsequently modelled in the finite element software DIANA FEA [11]. Both beams and columns were modelled as 'Class-III Beams 3D' elements, using their respective profile cross-section and elastoplastic steel material properties. During the modelling process, efforts were made to accurately represent the composite deck using a solid modelling approach, but the computational cost proved prohibitive. Therefore, a combination of linear and shell element geometries was employed to strike a balance between precision and computational cost. Regarding the supports, rigid supports were placed at the base of each column, fixing translations and rotations in the three directions.

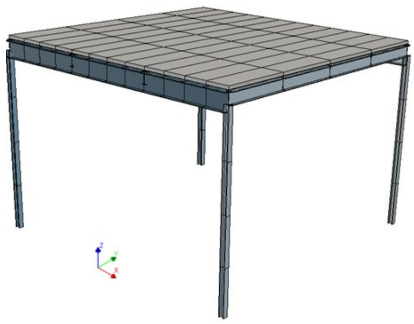

Figure 3. 3D view of a bay of the structure in DIANA FEA software.

Regarding the connections used in the modelling, fully rigid connections were employed throughout the building, except for the beam-to-column connections in the shorter direction (Y direction, see Figure 2 and Figure 3), which were modelled according to the arrangement specified during the design phase. This configuration results in weaker connections with limited flexural strength between columns and beams in the shorter direction, which were modelled as pinned to effectively disregard their flexural contribution. Once the geometry and connections were modelled, the properties of the model mesh were defined. For the beam elements, two-node, three-dimensional Class III beam elements were employed, while the shell elements were modelled using four-node quadrilateral isoparametric curved shell elements. For mesh sizing, each beam element was divided into four equal parts, except for the beams in the longer direction, which were divided into twelve equal parts. Regarding the shell elements, each was divided into forty-eight equal parts. As a result, the whole building comprises a total of 4662 elements and 15624 nodes.

3. Methodology

As depicted in previous sections, this article aims to present a practical solution to prevent the collapse of an existing structure in the event of the local failure of one or more columns on the lower floors by generating an alternative load path. This alternative load path is proposed to be created through the use of retrofitting beams located on the top floor of the building.

To identify the most disadvantageous, and henceforth 'critical,' local failure, thirteen 'damage scenarios' were proposed, involving the removal of one or two columns on the ground floor, as illustrated in Figure 4. Following recommendations from American guidelines [2,12,13], external columns near the middle of the short side, the middle of the long side, at the corner of the building, and adjacent to the corner were removed. Additionally, internal columns in critical locations were removed, and scenarios involving the removal of two columns were also considered.

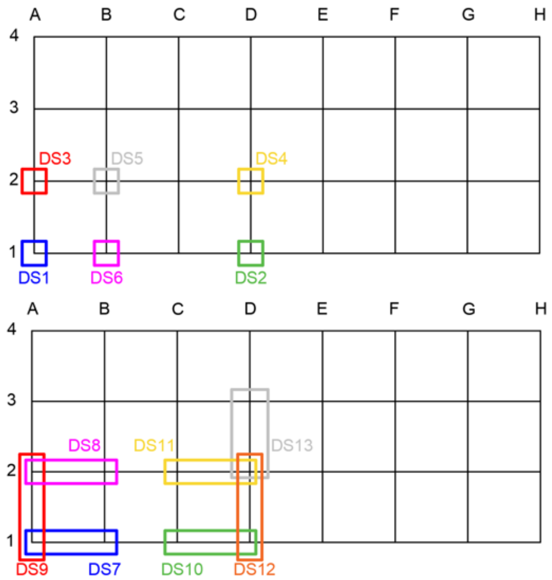

Figure 4. Proposed damage scenarios.

To evaluate the structural robustness of the building against each column removal, a computational methodology presented by Bao et al. [14] was employed. This methodology performs non-linear static pushdown analyses on reinforced concrete (RC) buildings under column loss scenarios, utilising an energy-based method (EBM) to account for the dynamic effects associated with sudden column loss. To verify the accuracy of the energy-based approach, the authors compared its results with those from direct dynamic analyses and found them to be closely aligned. Additionally, the EBM has been widely validated [14–17]. Xu and Ellingwood [15] even applied the EBM to assess the vulnerability of three steel frames to disproportionate collapse. They found that the predictions were closely aligned with the results of a nonlinear dynamic time history analysis. Therefore, the method proves useful for designing buildings with regular steel framing systems to resist disproportionate collapse.

The non-linear static pushdown analysis is therefore performed in the FEM software DIANA FEA for each damage scenario. The pushdown analysis procedure starts removing the column, or columns, and then loading all bays with the accidental loading combination G specified in Eurocode 0 [18]:

$$G = 1{,}0 * D + 0{,}7 * L \qquad (1)$$

where D is the dead load, and L is the live load. Following this, the adjacent bays on all floors above the removed column are incrementally loaded until the building collapses. The sum of the reactions on the adjacent columns to the removed one, under the peak vertical load intensity that the damaged structure can sustain following a sudden column loss (its ultimate capacity), is divided by the sum of the reactions under gravitational loads corresponding to the accidental load combination, resulting in a normalized ultimate capacity, λ_s.

Following the method proposed by Bao et al. [14], once the static load-displacement relationship is established, an energy-based analysis can be conducted to account for the dynamic effects associated with sudden column loss. According to energy principles, a pseudo-static normalized ultimate capacity λ_d is obtained. The energy-based method predicts that if $\lambda_d > 1$, the collapse will not occur. Bao et al. [14] also define a structure's robustness index as the minimum value of the normalized ultimate capacity over all damage scenarios.

Once the vulnerability of the different damage scenarios had been analysed, the retrofitting beam was studied. For this purpose, four retrofitting configurations on the top floor were proposed (Figure 5) and analysed to determine which one best enhances the building's behaviour and robustness. Additionally, non-linear dynamic analyses have been conducted to verify the accuracy of the results obtained from the energy-based method.

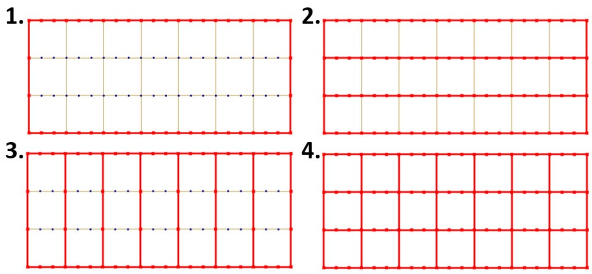

Figure 5. Proposed retrofit configurations.

4. Results and discussion

As mentioned in the previous section, thirteen damage scenarios were proposed, with six involving the removal of a single column and seven involving the removal of two columns. Figure 6 shows the pseudo-static normalized load intensity vs vertical displacement at the removed column relationship for all column removal scenarios. Additionally, Table 1 summarizes the results for each damage scenario.

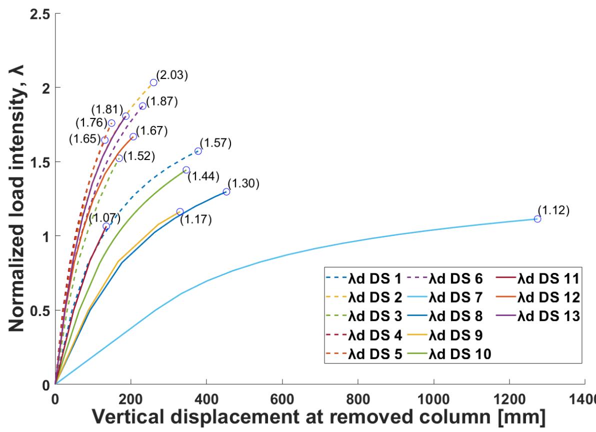

Figure 6. Pseudo-static results for the proposed damage scenarios before retrofit.

Table 1. Static and pseudo-static results for all damage scenarios before retrofit.

DS	λs	λd	Max. Vertical Displacement [mm]
1	2.01	1.57	378
2	2.70	2.03	260
3	2.26	1.52	169
4	2.37	1.65	131
5	2.54	1.76	149
6	2.57	1.87	231
7	1.40	1.12	1294
8	1.78	1.30	453
9	1.59	1.17	337
10	2.03	1.44	346
11	1.69	1.07	135
12	2.17	1.67	206
13	2.47	1.81	187

Analysing the results presented in Table 1, Damage Scenarios 7 and 11 showed low values of collapse resistance since their λ_d were the closest to 1. Therefore, the results indicate that if columns 2C and 2D (Damage Scenario 11) were to fail, the building would be on the verge of progressive collapse. For this case study, the authors decided on the safety side to set a minimum acceptable λ_d value of 1.10 to enhance the building's robustness. In the critical damage scenario, failure is triggered by yielding in the beams connecting the removed columns to the adjacent columns in the longitudinal direction. They undergo plastic deformation and eventually fail. This failure occurs near the adjacent column and is observed at all three levels of the structure (see Figure 7).

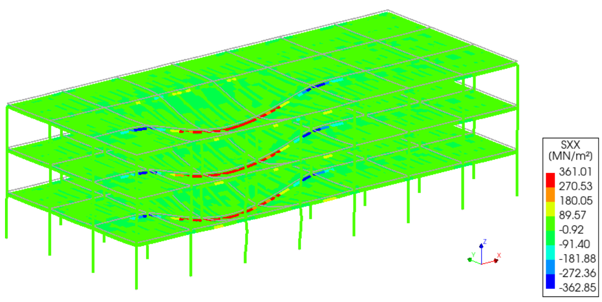

Figure 7. Structure's total stresses on direction 'X' for damage scenario 11 just before failure.

Once the 'critical' damage scenario was identified, the retrofitting beam configurations (Figure 5) are proposed to determine the most appropriate configuration. Regarding the design of the retrofitting beams, five standard profiles were considered for each beam configuration (IPE 270, IPE 300, IPE 330, IPE 450 and IPE 600). The retrofitting beams were modelled directly above the roof-level floor's existing beams, assuming a perfect bond between the retrofit and the existing beams. For simplicity, any pre-existing deformation of the original beams was disregarded, and the analyses were initiated using the reinforced models. The results of these analyses are presented in Table 2 and Table 3. If the decision were based solely on increasing the robustness of the worst damage scenario, DS11, the best option would be between configurations 2 and 3, which exceed the λ_d value of 1.10 with an IPE 270 profile. However, considering both column removal scenarios, configuration 3, particularly with the IPE 300 beam size, proved to be the better choice. This configuration increased the λ_d values from 1.12 and 1.07 (for DS7 and DS11, respectively) to 1.13 and 1.17, surpassing the established minimum value of 1.10 while incurring relatively low costs compared to other configurations. Furthermore, when evaluating Configurations 2 and 3 with an IPE 300 profile, performance under Damage Scenario 7 was a decisive factor, as Configuration 2 failed to deliver satisfactory results. It is seen that Configuration 3 outperforms Configuration 2, as it reinforces the weaker direction of the building, where beams are discontinuous and modelled with pinned connections. This configuration ensures the usual moment transfer between beams in the stronger direction while adding moment transfer in the weaker direction through the retrofitting beams. While the failure mode remains unchanged after the retrofit for the critical damage scenarios, failure occurs at significantly higher load levels.

Additionally, configuration 3 with IPE 300 profiles was evaluated for the remaining column removal scenarios (Table 4 and Figure 8), showing that it either improved or at least did not impair the building's performance against progressive collapse in all cases except for DS10, which exhibits a slight decrease in λ_d. However, the λ_d value remains much higher than the minimum acceptable level and the results reveal a notable reduction in maximum displacement under higher loads. This indicates that the selected reinforcement effectively enhances the stiffness of the structure. Consequently, its inclusion proves to be a beneficial measure. As a result, this solution was selected for meeting the design objectives at a lower cost than the other alternatives.

Table 2. Analyses results combining configurations and profile sizes for DS7.

Parameter		Damage Scenario 7 Configuration 1	2	3	4
No Retrofit	λs	1.40			
	λd	1.12			
	Displacement [mm]	1294			
IPE 270	λs	1.51	1.45	1.50	1.51
	λd	1.13	1.05	1.09	1.11
	Displacement [mm]	656	536	558	580
IPE 300	λs	1.53	1.48	1.54	1.53
	λd	1.13	1.05	1.13	1.12
	Displacement [mm]	595	492	544	528
IPE 330	λs	1.57	1.51	1.62	1.61
	λd	1.15	1.08	1.19	1.19
	Displacement [mm]	568	480	557	550
IPE 450	λs	1.67	1.71	1.85	1.82
	λd	1.17	1.23	1.32	1.28
	Displacement [mm]	362	400	409	374
IPE 600	λs	2.05	2.07	2.37	2.37
	λd	1.40	1.42	1.63	1.62
	Displacement [mm]	276	283	309	305

Table 3. Analyses results combining configurations and profile sizes for DS11.

Parameter		Damage Scenario 11 Configuration 1	2	3	4
No Retrofit	λs	1.69			
	λd	1.07			
	Displacement [mm]	135.1			
IPE 270	λs	1.71	1.78	1.81	1.92
	λd	1.08	1.14	1.15	1.24
	Displacement [mm]	138	140	140	150
IPE 300	λs	1.71	1.82	1.84	1.97
	λd	1.09	1.16	1.17	1.27
	Displacement [mm]	138	143	141	150
IPE 330	λs	1.71	1.87	1.88	2.03
	λd	1.08	1.21	1.20	1.32
	Displacement [mm]	138	151	142	153
IPE 450	λs	1.72	2.03	2.08	2.42
	λd	1.09	1.33	1.34	1.62
	Displacement [mm]	138	162	145	176
IPE 600	λs	1.73	2.14	2.42	2.70
	λd	1.10	1.37	1.59	1.70
	Displacement [mm]	138	143	145	128

Finally, non-linear dynamic analyses for the critical scenarios under the accidental loads combination with and without the stiffening beams were conducted to verify the accuracy of the results obtained from the energy-based method. Figures 9 and 10 illustrate the difference in the dynamic response with and without retrofit for Damage Scenarios 7 and 11, respectively. The vertical axis represents the vertical displacement measured at the removed column where this displacement is greatest. The horizontal axis represents time. Lastly, Figures 11 and 12 present the static, pseudo-static, and direct dynamic responses for Damage Scenario 7 and Damage Scenario 11, with and without the selected beam configuration.

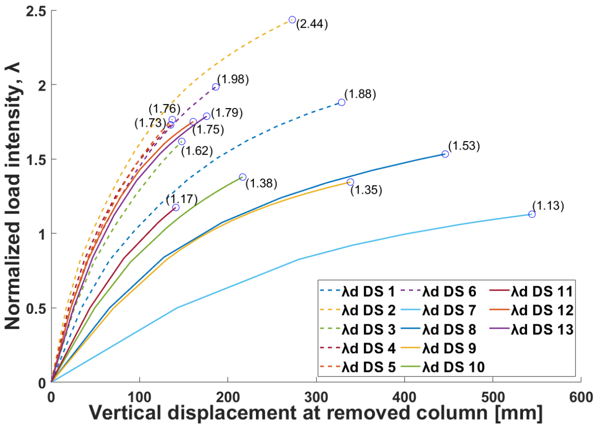

Figure 8. Pseudo-static results for the proposed damage scenarios with the selected retrofit configuration.

Table 4. Analyses results for the proposed damage scenarios with the selected retrofit configuration.

DS	λs	λd	Max. Vertical Displacement [mm]
1	2.48	1.88	328.8
2	3.21	2.44	272.9
3	2.47	1.62	147.8
4	2.50	1.73	135.1
5	2.60	1.76	137.1
6	2.79	1.98	186.3
7	1.54	1.13	544.2
8	2.05	1.53	446.0
9	1.81	1.35	338.7
10	2.05	1.38	216.8
11	1.84	1.17	140.9
12	2.37	1.75	160.8
13	2.48	1.79	176.0

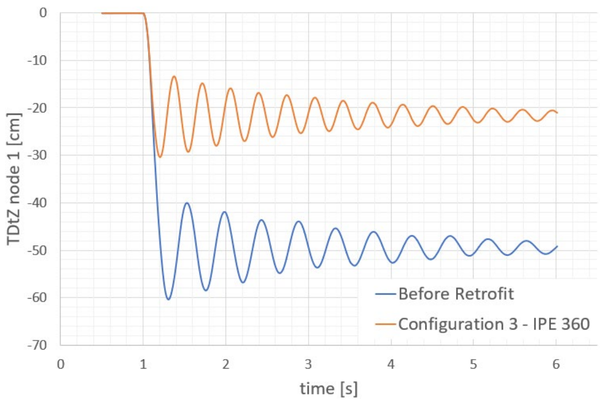

Figure 9. Direct dynamic analyses responses for Damage Scenario 7 with and without retrofit.

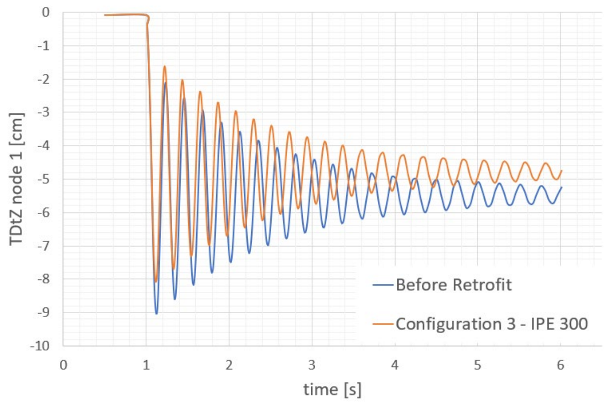

Figure 10. Direct dynamic analyses responses for Damage Scenario 11 with and without retrofit.

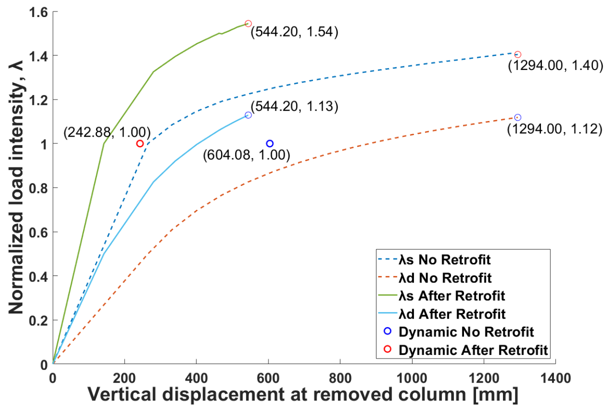

Figure 11. Static, pseudo-static, and dynamic responses for Damage Scenario 7.

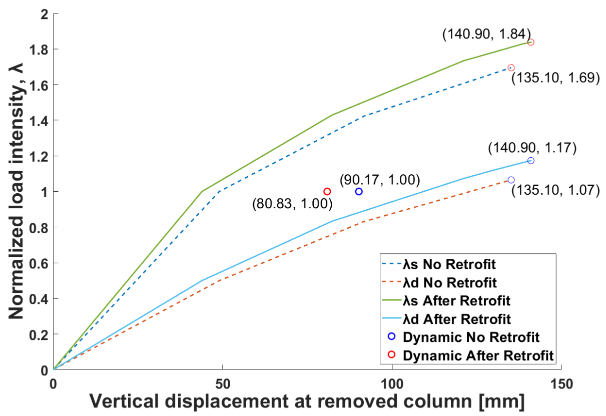

Figure 12. Static, pseudo-static, and dynamic responses for Damage Scenario 11.

5. Conclusions

This article investigates the effectiveness of a novel retrofitting technique, which involves placing roof-level retrofitting beams to enhance resistance to progressive collapse, on a three-story steel building. The structure was designed according to Spanish standards and then modelled in the finite element software DIANA FEA. To evaluate the structural robustness, thirteen damage scenarios were proposed, simulating the collapse of one or two columns. To analyse the vulnerability of the proposed damage scenarios an energy-based method was employed. A number of possible retrofit configurations were analysed and the most effective was selected. The results obtained from the energy-based method were validated through non-linear dynamic analyses to ensure the reliability of the proposed solution. The results indicate that using configuration 3 (perimeter beams plus cross beams) with the IPE 300 profile provides an effective solution, offering a balance between enhanced robustness and cost efficiency. The study led to the following conclusions:

- The selected roof-level retrofitting beam configuration enhances the building's robustness, increasing the λ_d value for the critical damage scenario from 1.07 to 1.17.
- While the failure mode remains unchanged after adding the stiffening beams, failure occurs at significantly higher load levels.
- The retrofitting beams effectively redistribute gravity loads, involving the entire structure in the load distribution after column removal, demonstrating it as a global retrofit solution.
- The perimeter arrangement of the retrofitting beam is ineffective against internal column damage scenarios.
- The anisotropic nature of the building regarding its beam connections causes the optimal stiffening configuration to necessarily include reinforcement of the structure's "weak" direction.
- The methodology proposed by Bao saves computational costs in a reliable way for analysing vulnerable scenarios as it allows direct comparison without having to iterate between different load states as we would in the dynamic analysis.

Acknowledgements

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