European Community on Computational Methods in Applied Sciences

ECCOMAS Thematic Conference

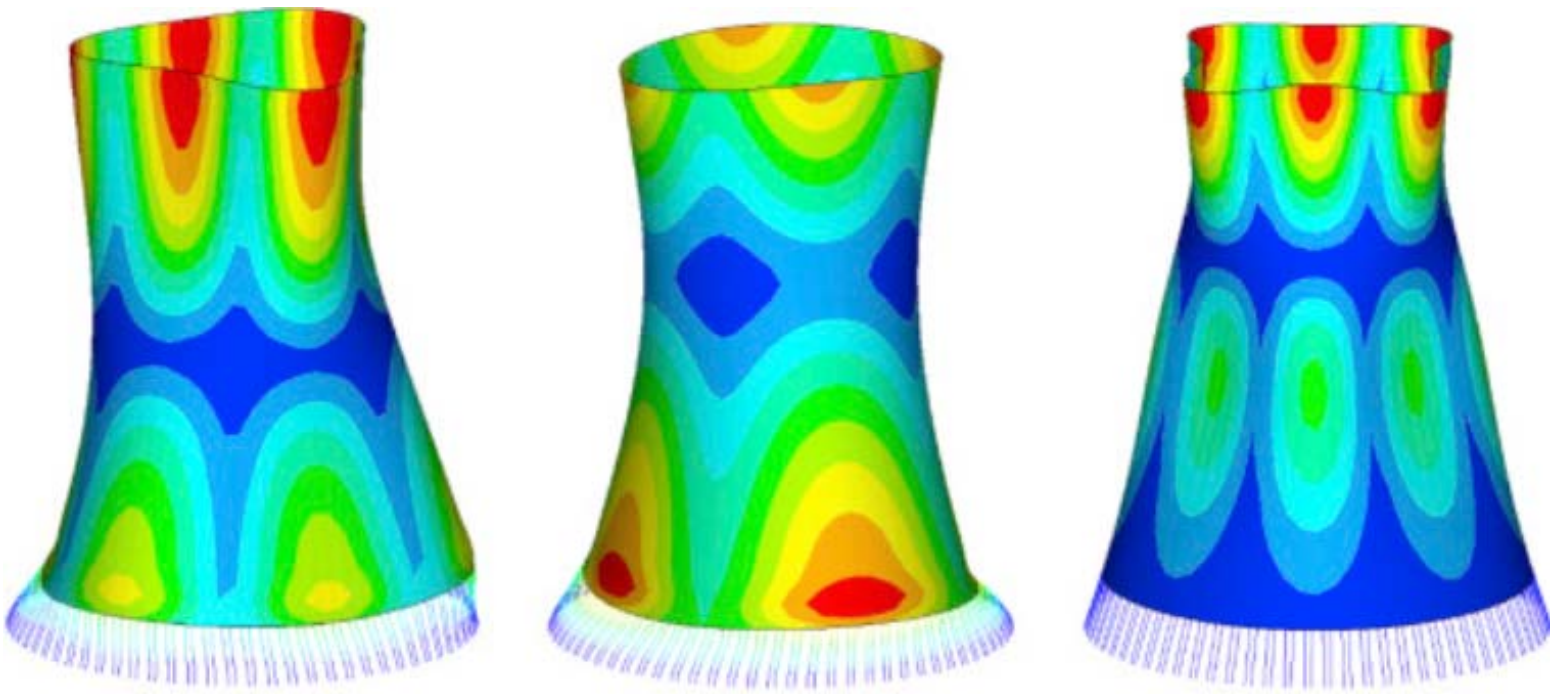
COMPDYN 2019

7th International Conference on Computational Methods in Structural Dynamics and Earthquake Engineering

PROCEEDINGS

Volume II

M. Papadrakakis, M. Fragiadakis (Eds)



An IACM Special Interest Conference

COMPDYN 2019
Computational Methods in Structural Dynamics and Earthquake Engineering

Proceedings of the 7th International Conference on Computational
Methods in Structural Dynamics and Earthquake Engineering
Held in Crete, Greece
24-26 June 2019

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A publication of:

Institute of Structural Analysis and Antiseismic Research
School of Civil Engineering
National Technical University of Athens (NTUA)
Greece

COMPDYN 2019

Computational Methods in Structural Dynamics and Earthquake Engineering

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First Edition, October 2019

ISBN (set): **978-618-82844-5-6**
ISBN (vol II): **978-618-82844-7-0**

PREFACE

This volume contains the full-length papers presented in the 7th International Conference on Computational Methods in Structural Dynamics and Earthquake Engineering (COMPDYN 2019) that was held on June 24-26, 2019 in Crete, Greece.

COMPDYN 2019 is one of the 32 Thematic Conferences of the European Community on Computational Methods in Applied Sciences (ECCOMAS) to be held in 2019 and is also a Special Interest Conference of the International Association for Computational Mechanics (IACM). The purpose of this Conference series is to bring together the scientific communities of Computational Mechanics, Structural Dynamics and Earthquake Engineering, to act as the forum for exchanging ideas in topics of mutual interests and to enhance the links between research groups with complementary activities. We believe that the communities of Structural Dynamics and Earthquake Engineering will benefit from their exposure to advanced computational methods and software tools which can highly assist in tackling complex problems in dynamic and seismic analysis and design, while also giving the opportunity to the Computational Mechanics community to be exposed to very important engineering problems of great social interest. The COMPDYN 2019 Conference is supported by the National Technical University of Athens (NTUA), the European Association for Structural Dynamics (EASD), the European Association for Earthquake Engineering (EAEE), the Greek Association for Computational Mechanics (GRACM).

The editors of this volume would like to thank all authors for their contributions. Special thanks go to the colleagues who contributed to the organization of the Minisymposia and to the reviewers who, with their work, contributed to the scientific quality of this e-book.

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ACKNOWLEDGEMENTS

The conference organizers acknowledge the support towards the organization of the "7th International Conference on Computational Methods in Structural Dynamics and Earthquake Engineering", to the following organizations:

- European Community on Computational Methods in Applied Sciences (ECCOMAS)
- European Association for Structural Dynamics (EASD)
- European Association for Earthquake Engineering (EAEE)
- Greek Association for Computational Mechanics (GRACM)
- Hellenic Society for Earthquake Engineering (HSEE)
- School of Civil Engineering, National University of Athens (NTUA)
- Hellenic Republic-Region of Crete
- Municipality of Heraklion

Plenary Speakers and Invited Session Organizers

We would also like to thank the Plenary and Semi-Plenary Speakers and the Minisymposia Organizers for their help in the setting up of a high standard Scientific Programme.

Plenary Speakers: Michel Bruneau, Álvaro Cunha, Michael Fardis, Charbel Farhat, Christian Soize, Alexander Vakakis

Semi-Plenary Speakers: Christoph Adam, Gian Paolo Cimellaro, Eleni Chatzi, Geert Degrande, Boris Jeremic, Shinobu Yoshimura

MS Organizers: Günther Achs, Christoph Adam, Hamid Ahmadi, António Arêde, Rui Carneiro Barros, Michael Beer, Amadeo Benavent-Climent, Manuel Braz-Cesar, Alessandro Cabboi, Silvia Caprili, Claudia Casapulla, Serena Cattari, Nicola Cavalagli, Liborio Cavaleri, Jianbin Chen, Dimitris Chronopoulos, Alice Cicirello, Gian Paolo Cimellaro, Francesco Clementi, Marco Corradi, Flavia De Luca, Carlo Del Gaudio, Ciro Del Vecchio, Pedro Delgado, Raimundo Delgado, Raffaele Di Laora, Marco Di Ludovico, Luigi Di Sarno, Fabio Di Trapani, Francisco Alejandro Diaz de la O, Marco Domaneschi, Stefanos Dritsos, Hossein Ebrahimian, Antonio Formisano, Michalis Fragiadakis, Linda Giresini, Jose Gonzalez, Alexander Idesman, Alper Ilki, Maria Iovino, Fatemeh Jalayer, Hector Jensen, Andreas Kappos, Jin-Gyun Kim, Radek Kolman, Davide Lavorato, Guido Magenes, Charilaos Maniatakis, George Manos, Gabriele Milani, Stergios Mitoulis, Naoto Mitsume, Fabrizio Mollaioli, Paolo Morandi, Lukas Moschen, George Mylonakis, Jiri Naprstek, Ehsan Noroozinejad, Camillo Nuti, Roger Ohayon, Georgios S. Papavasileiou, K.C. Park, Vagelis Plevris, Nikos G. Pnevmatikos, Maria Polese, Laura Ragni, Andrei M. Reinhorn, Paolo Ricci, Hugo Rodrigues, Emmanouil Rovithis, Juan Chiachío Ruano, Manuel Chiachío Ruano, Walter Salvatore, Fabrizio Scozzese, Anastasios Sextos, Castorina Silva Vieira, Andrei L. Smirnov, Sergey Sorokin, Enrico Spacone, Daniele Spina, Constantine Spyrakos, Francesca Taddei, Anton Tkachuk, Petr Evgen'evich Tovstik, Savvas Triantafyllou, Yiannis Tsompanakis, Enrico Tubaldi, Marcos Valdebenito, Humberto Varum, Ioannis Vayas, Gerardo Mario Verderame, Stefania Viti, Shinobu Yoshimura

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Crete, Greece, 24–26 June 2019

COMPUTATIONAL ISSUES OF HINGED WALLS USED AS RETROFITTING OF EXISTING RC FRAMES

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Abstract

In recent years hinged walls have been implemented as a retrofit technique for existing RC buildings. To investigate the effectiveness of the proposed solution on different frame typologies, non-linear 2D pushover analyses have been carried out. Two main configurations were adopted, representing an inner frame with weak beams and strong columns and a side frame with strong beams and weak columns, respectively. The study shows that some computational aspects are of fundamental importance in providing reliable results, namely: the dead load distribution on the beams and the moment-axial force interaction in the columns. The hinged wall technique proves to be an effective retrofit solution only if conceived properly for each structural typology; whilst in some cases it may be detrimental when applied in the traditional way. Some new configurations are herein proposed based on new connection layouts in order to be suitable for the different typologies of existing RC frames.

Keywords: Existing RC structures, retrofit techniques, hinged walls, computational issues.

1 INTRODUCTION

The poor state of conservation of the building heritage requires a deep renovation action, considering that 40% of it has already exhausted its nominal service life, and most of existing buildings are obsolete and vulnerable to seismic actions. In detail, the post Second World War reinforced concrete heritage represents the 60% of such building stock (Belleri & Marini, 2016 [1]; Marini et al., 2017 [2]; Labò et al., 2017 [3]; Belleri et al., 2016 [4]; Feroldi et al., 2013 [5]). Those buildings are typically multi-storey structures, featuring one-way reinforced concrete (RC) frames, designed for gravity loads, with inadequate structural details. These features highly contribute to increase their seismic vulnerability, which is typically associated with the possible onset of soft-storey mechanisms or brittle failure of short columns. Hinged walls represent a possible retrofit solution, aiming at linearizing the deformation of the frame along its height (Mac Rae et al., 2004 [6]). Such linearization entails a more uniform deformation demand in frame stories and thus a more predictable damage pattern. The proposed system can be used both in existing and new structures: it can be connected to the existing building from outside, or even some existing elements can be adapted to this function, for example the stairwell walls. In the last few years, application of hinged walls from outside as retrofit solution has increased (Wada et al., 2011 [7]; Gioiella et al., 2017 [8]), and it could be even more widely used in the future. The present paper investigates the suitability of hinged wall solution in the seismic retrofit of existing RC buildings. In particular, the role of the beam-column capacity ratio and the number of effective links are investigated by means of nonlinear static analyses.

2 APPLICATION TO AN EXISTING BUILDING

Since the application of hinged walls in existing buildings as a retrofit solution is getting attention worldwide, the weaknesses and the possible beneficial effects of their application need to be further investigated. On the whole, hinged walls are conceived as external walls connected to the existing building by means of links at each floor level; the wall is usually designed to remain in the elastic field during a target seismic event. The crucial difference between the proposed solution and the more widespread rocking wall is the presence of an explicit pinned constraint at the base, which allows to reduce the bending moment transferred to the foundation. In the following, the hinged wall is implemented in some typical configurations of RC existing buildings, represented by 2D frames.

2.1 Design of the existing building

For the sake of simplicity, the reference frame is assumed to be regular both in plant and elevation. The geometrical scheme of the frame is represented in Figure 1; geometrical and mechanical characteristics of beams and columns are assumed to be homogeneous along the height of the building. A concrete cylindrical strength of 30MPa and steel yielding stress of 450MPa are assumed. The frame has five floors and five bays, with a total dimension and height of 22.5 m and 15 m, respectively. In existing buildings, two main structural configurations are generally possible: a frame with weak beams and strong columns or a frame with strong beams and weak column. Here, the inner longitudinal frame (case A) and the side (case B) longitudinal frame of a building are analysed to describe the aforementioned typologies, respectively. The structural elements are designed according to the Italian building code enforced in the ‘60s, with reference to the “admissible stresses method”. Structural details such as main dimensions, and bottom (A_{sb}) and top (A_{st}) rebar are reported in Table 1 and Table 2.

Once the two principal configurations have been defined, two more frames are modelled fostering their characteristics to obtain a frame with very weak beams and very strong col-

umns (case C) and the opposite (case D). This further investigation allows a better understanding of the influence of the frame configuration on the response of the retrofitted building. So, the four models used in the following analyses are:

- CASE A: Existing inner frame: weak beams, strong columns
- CASE B: Existing side frame: strong beams, weak columns
- CASE C: Modified inner frame: weak beams, strong columns
- CASE D: Modified side frame: strong beams, weak columns

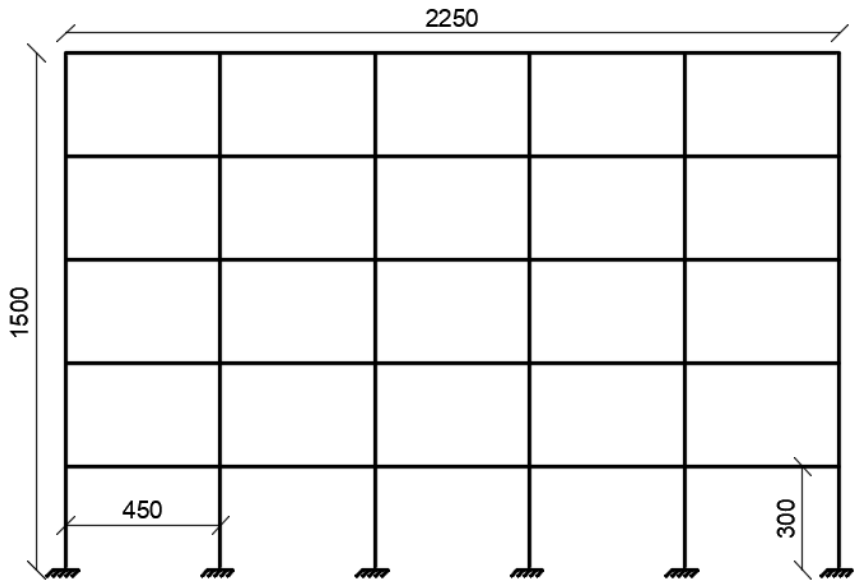

Figure 1: 2D Frame geometrical configuration (dimensions in cm)

In order to understand the influence of the beams and columns capacity, the "nodal ratios" are computed for each frame type, as the ratio between the sum of the ultimate resisting moment of the beams and the sum of the ultimate resisting moment of the columns converging in a node (Table 3). Given the layout of the nodes, the mean nodal ratio is calculated for each floor.

Table 1: Reinforcement of beams

BEAM DETAILS				Beam end			Beam centre		
CASE	b (cm)	h (cm)	c (cm)	A_{sb}	A_{su}	Stirrups	A_{sb}	A_{su}	Stirrups
A	80	24	3	3φ16	9φ16	φ 10/10 cm	6φ16	3φ16	φ 10/250 cm
B	30	35	3	3φ16	4φ16	φ 10/10 cm	3φ16	3φ16	φ 10/250 cm
C	80	24	3	3φ16	6φ16	φ 10/10 cm	4φ16	3φ16	φ 10/250 cm
D	30	50	3	3φ16	4φ16	φ 10/10 cm	3φ16	3φ16	φ 10/250 cm

Table 2: Reinforcement of columns

CASE	b (cm)	h (cm)	c (cm)	A_{sb}	A_{su}	Stirrups
A	30	30	3	2φ16	2φ16	φ 8/30 cm
B	30	30	3	2φ16	2φ16	φ 8/30 cm
C	30	30	3	2φ20	2φ20	φ 8/30 cm
D	30	30	3	2φ16	2φ16	φ 8/30 cm

Table 3: Nodal Ratios

CASE	FLOOR 1	FLOOR 2	FLOOR 3	FLOOR 4	FLOOR 5
A	0.75	0.80	0.92	1.12	2.57
B	0.77	0.86	0.98	1.17	2.60
C	0.45	0.48	0.53	0.62	1.38
D	1.13	1.26	1.44	1.71	3.82

Concerning the static loads on the structure, a permanent structural and non-structural load (4 kN/m^2 and 2 kN/m^2, respectively) and a live load of 2 kN/m^2 are considered. In addition, a linear load of 6 kN/m has been included in the side frame to account for perimetral infills. The actual stiffness of beams and columns has been reduced to 50% and 70% of the initial value respectively, to account for concrete cracking. The structure has been modelled trough the software MidasGEN [9].

2.2 Design of the retrofit intervention

The analysis of the effects of hinged wall solution is the main objective of the present study. Typically, the main design parameter for hinged walls is the stiffness required to linearize the deformation of the frame along its height. A hinged wall to frame storey lateral stiffness ratio is identified as α (1):

$$\alpha = \frac{E_w J_w}{K_s H^3} \tag{1}$$

where the H is the structure total height, K_s is the frame storey approximate lateral stiffness, E_w and J_w are the Young modulus and the moment of inertia of the hinged wall, respectively. Previous studies (Mac Rae et al., 2004 [6]) have demonstrated that α equal to 0.1 is the optimal value required to linearize the frame deformation; beyond such value, there is no beneficial effects from increasing the dimensions of the wall. Concerning the estimation of the frame storey lateral stiffness, the approximated method proposed by Schultz, 1992 [10] has been adopted. The method was derived for regular frames, fixed at the base, and accounting only for flexural deformations. The following simplified expression is used in the present study:

$$K_s = \left(\frac{24}{h_c^{\,2}}\right)\left(\frac{1}{\frac{2}{\sum k_c} + \frac{1}{\sum k_{ga}} + \frac{1}{\sum k_{gb}}}\right) \tag{2}$$

where h_c is the storey height, $\sum k_c$ the sum of the stiffness of the column in a given storey, $\sum k_{ga}$ and $\sum k_{gb}$ the sum of the flexural stiffness of the girders framing into the joint above and below the columns, respectively. The stiffness of each member (column or girder) is assumed as $k=EJ/L$. So, the hinged wall moment of inertia can be obtained from the former definition of the ratio α (1); then, the ratio between the wall dimensions is assumed herein as $b=0.15h$ and the height of the wall section is calculated (3). The dimensions of the hinged walls for the four frames are summarized in Table 4.

$$h = \sqrt[4]{\frac{12 J_w}{0.15}} \tag{3}$$

Table 4: Hinged wall dimensions required to linearize the frame deformation

CASE	b (cm)	H (cm)
A	27	180
B	28	190
C	27	180
D	32	210

3 COMPUTATIONAL ISSUES

The nonlinear static (pushover) analyses carried out on the frames have shown significant results concerning some computational aspects of the modelled frames. In particular, it is important to include the distributed loads on the beams and to account for the moment-axial forces interaction in the columns to capture the global system performance. In addition, connecting the hinged wall at each floor is not always beneficial.

3.1 Columns Design and Load distribution

The frame described in case A is here implemented to explain some relevant computational aspects: the analyses carried out show how the building modelling affects the response of the structure. In detail, it is important to take into account the actual distribution of the gravity loads on the beams: indeed, the distribution of gravity loads produces an initial rotation at the beam ends, which can significantly reduce the residual deformation capacity of the retrofitted building. The hinged wall, aiming at linearizing the deformation of the building along its height, produces an additional large rotation in the beams and may cause the premature beam failure. Besides, lumping the distributed loads into point loads applied at the nodes produces an overestimation of the system deformation capacity. Such results are summarized in Figure 2 and Figure 3, where the base shear-displacement graph for each modelled frame is reported.

The other crucial aspect to be considered is the appropriate definition of the axial-flexural interaction for RC columns. When considering such interaction (PM_int), a substantial increase in the capacity of the building is recorded; it is worth noting that the detrimental effect highlighted before (i.e. when distributing the gravity loads on the beams) is observed only when such interaction is considered.

The beneficial effect of the hinged wall application can be seen as an increase in both capacity and moreover ductility of the retrofitted system, while the opposite effects are considered to be detrimental. For instance, in Figure 2 it is clear that for the same structure, the hinged wall solution appears to be detrimental if considering the axial-flexural interaction (A_HW_PM_int) for columns and beneficial without considering it (A_HW_no_PM_int), with respect to the corresponding as-is case (corresponding dotted lines). At the same time, such detrimental effect is not detected when loads are modelled as point loads and lumped at the nodes (Figure 3): in this case, the hinged wall solution appears to be always beneficial, even though the only increase in ductility is detected by accounting for axial-flexural interaction.

So, in this specific case, it is possible to observe the detrimental effect for the system only when accounting for both the distributed loads on the beams and the axial-flexural interaction for the columns; beside the mentioned example, the results show that in any case the computational aspects need to be carefully defined in order to have a reliable estimation of the structural behaviour and so an optimal design of the retrofit solution.

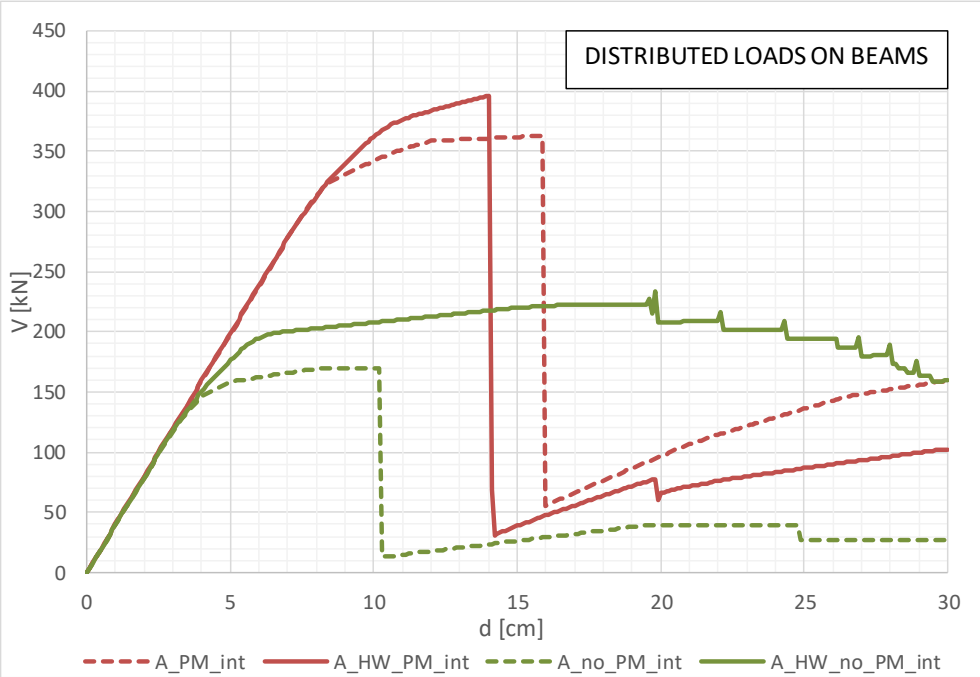

Figure 2: Models comparison in the case of distributed dead loads on the beams: A=Frame in Case A, HW=Hinged Wall as a retrofit solution, PM_int/no_PM_int = accounting or not for the Moment-Axial forces interaction in the columns.

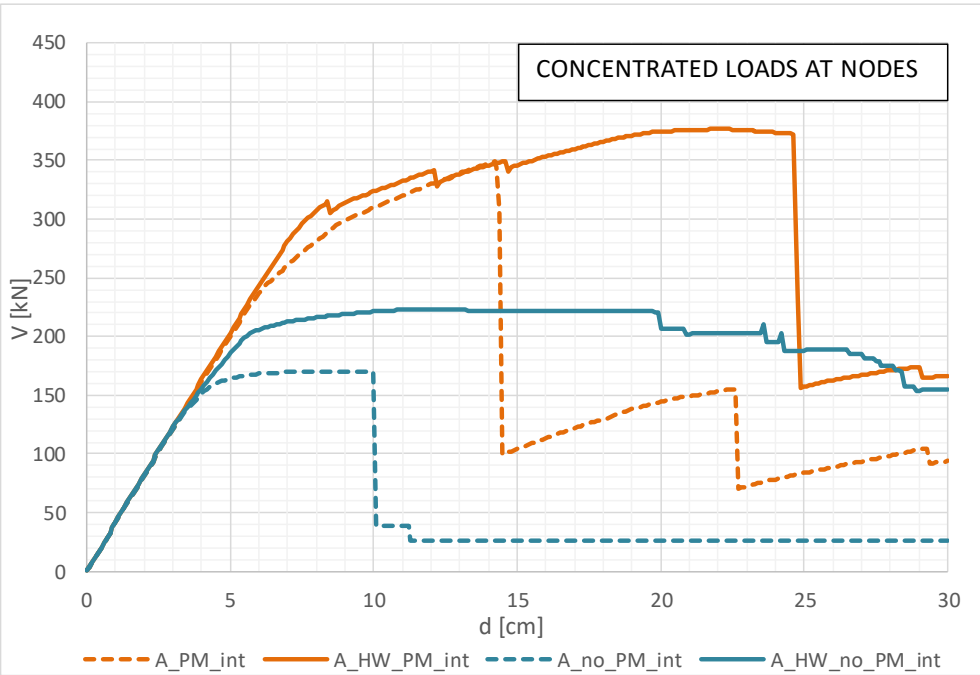

Figure 3: Models comparison in the case of concentrated dead loads at the nodes: A=Frame in Case A, HW=Hinged Wall as a retrofit solution, PM_int/no_PM_int = accounting or not for the Moment-Axial forces interaction in the columns.

3.2 Links Configuration

The previous results show that in some cases the high deformation demand due to the introduction of the hinged wall can be critical for the structural elements. Each link controls the displacement of the associated storey, forcing the elements deformation.

A possible solution is thus investigated by modifying the number and configuration of links along the building height. The results highlight the influence of the nodal ratios of the frames in the effectiveness of the retrofit solution. When dealing with a "weak beams – strong columns" frame type, the imposed displacement profile leads to a deformation demand for beams that is far larger than their capacity, leading to their premature failure. Such results are addressed herein. First, all the considered link configurations are shown in Figure 4, then the pushover curves are reported for each configuration and the behaviour of the frames before collapse highlighted (Figure 5 - Figure 12).

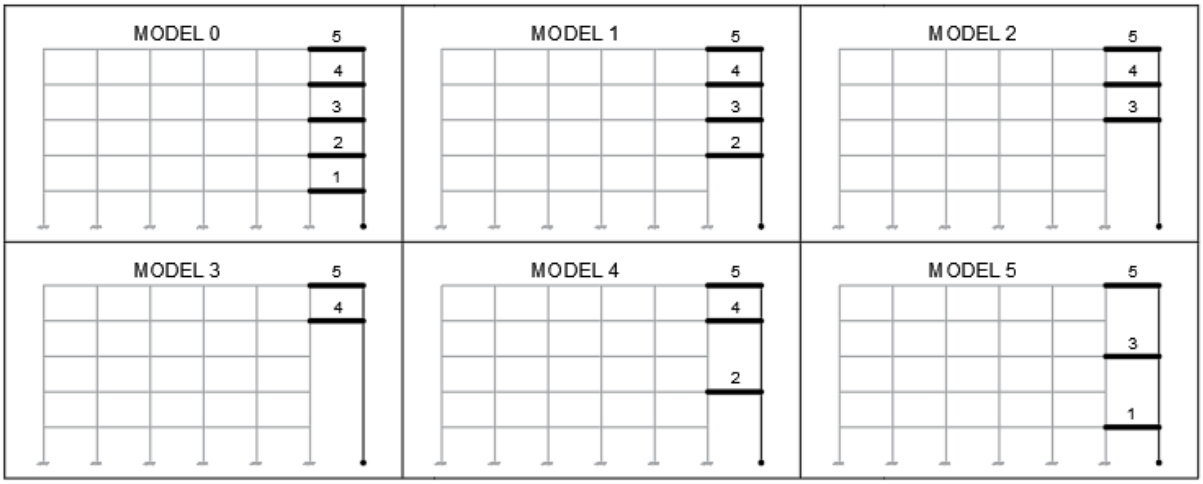

Figure 4: Links configuration considered in the models

The following graphs show the comparison between the retrofitted building response in the case of different configurations of the links for each frame type; the model "as is" represents the building before retrofit. The results show that the effectiveness of the traditional hinged wall solution (i.e. connecting the wall to the frame at each floor) is substantial in the case of "modified side frame", while the adoption of hinged walls is detrimental in the case of the "modified inner frame". This is associated with the linearization of the deformation along the height, which is particularly useful in the case of possible onset of soft storey mechanisms, and so in the case of strong beams and weak columns. Such linearization prevents the collapse of the columns in a soft storey mechanism. In the other cases, as explained before, the imposed linearization causes a rotation demand in some beams which exceeds their rotation capacity, and so a detrimental effect is observed.

The nodal ratios of the frame stories seem to be closely related to the collapse behaviour of the different frames: the larger are the nodal ratios and the more effective is the application of the hinged wall in the traditional way, i.e. connected at each floor. For this reason, nodal ratios are relevant parameters to be investigated for the retrofit solution with hinged walls.

To overcome the possible detrimental effects observed, different configurations of the links were investigated: for each frame type, the best solution was identified as the one that produced the highest deformation capacity of the system.

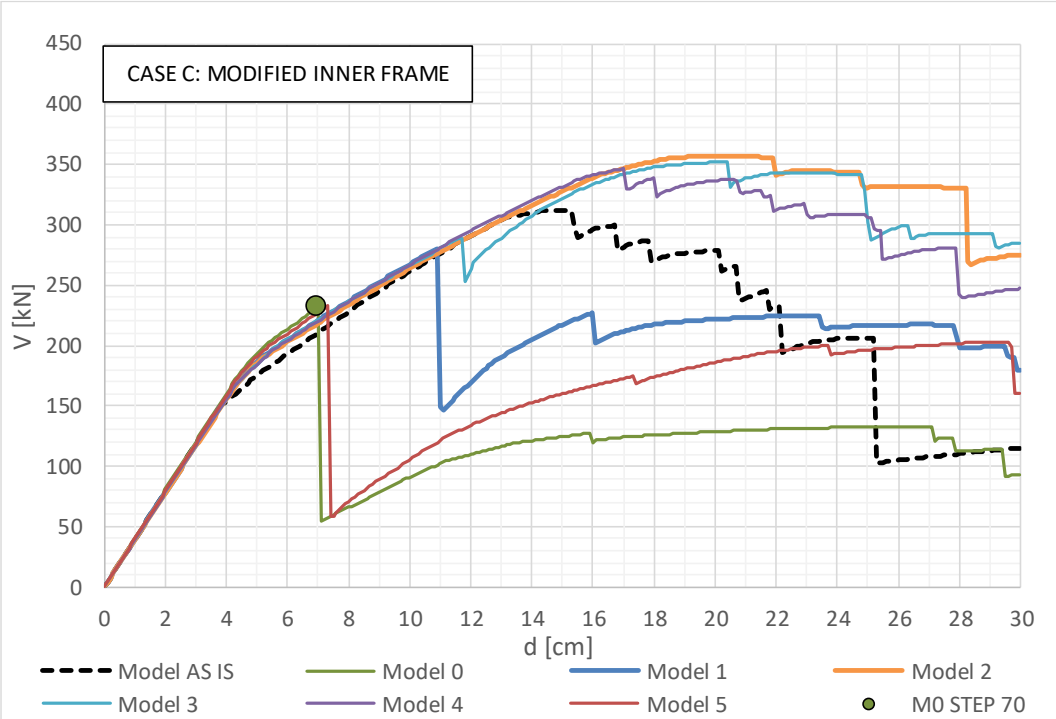

Figure 5: Comparison of pushover results for the frame C different configurations: M0 STEP 70 (7 cm) represents the hinge status result described in Figure 6. Optimal solution: Model 2.

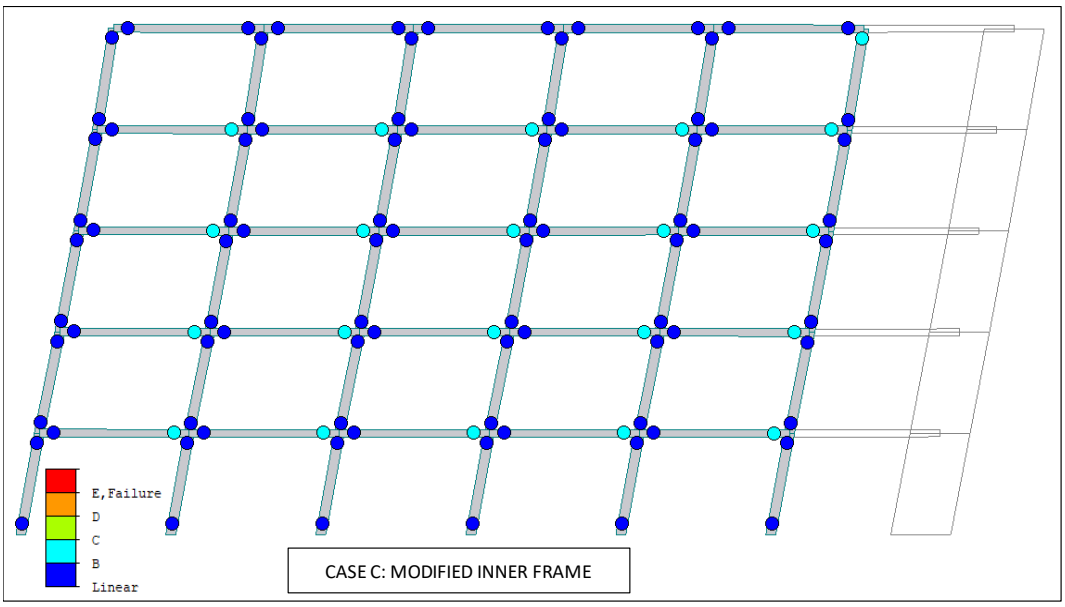

Figure 6: Plastic hinge status result at the step before collapse of frame C in the case of Hinged Wall connected at each floor – Model 0 STEP 70 (7 cm) in Figure 5.

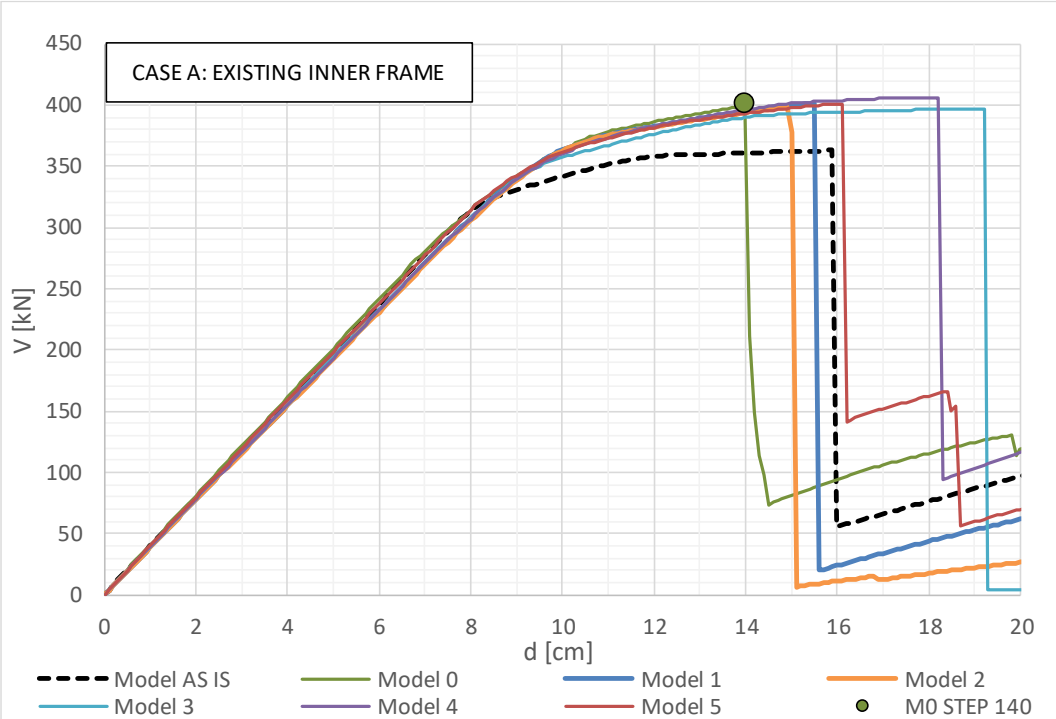

Figure 7: Comparison of pushover results for the frame A different configurations: M0 STEP 140 (14 cm) represents the hinge status result described in Figure 8. Optimal solution: Model 3.

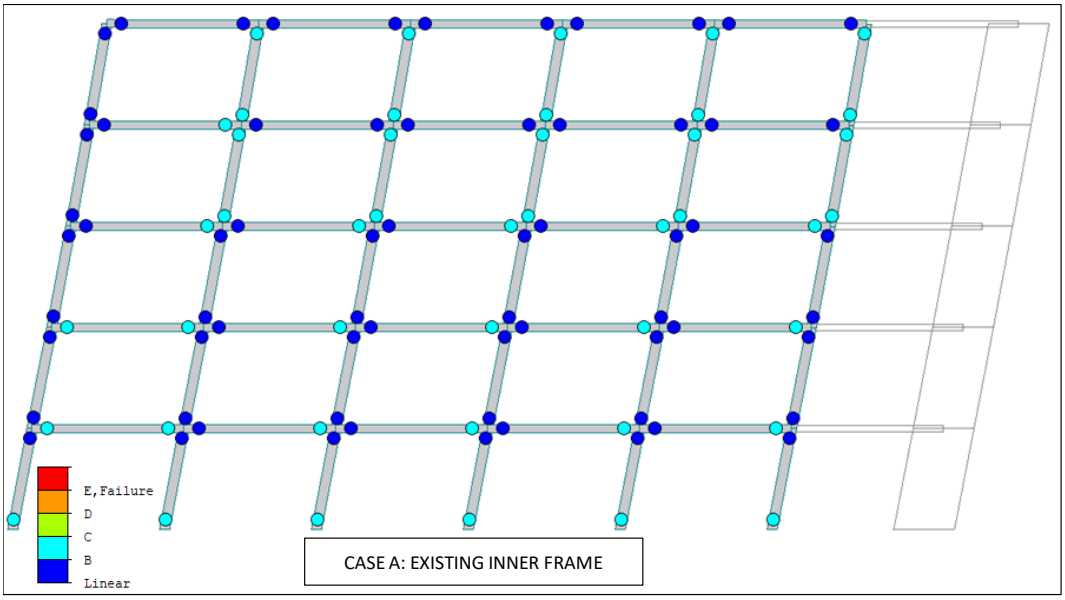

Figure 8: Plastic hinge status result at the step before collapse of frame A in case of Hinged Wall connected at each floor – Model 0 STEP 140 (14 cm) in Figure 7.

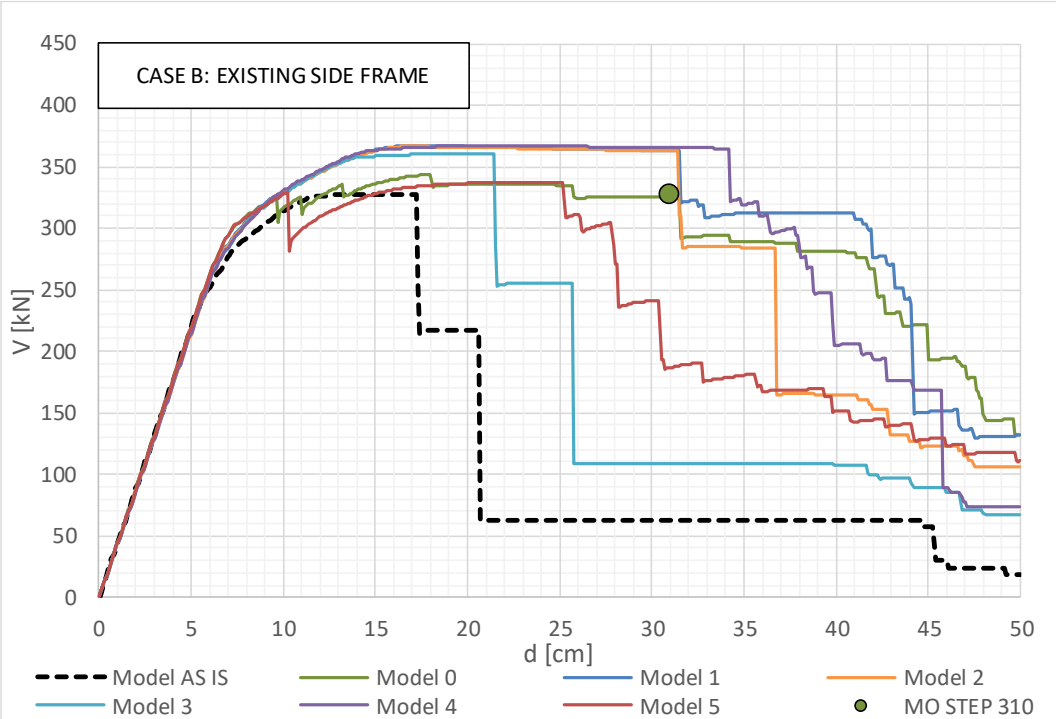

Figure 9: Comparison of pushover results for the frame B different configurations: M0 STEP 310 (31 cm) represents the hinge status result described in Figure 10. Optimal solution: Model 4.

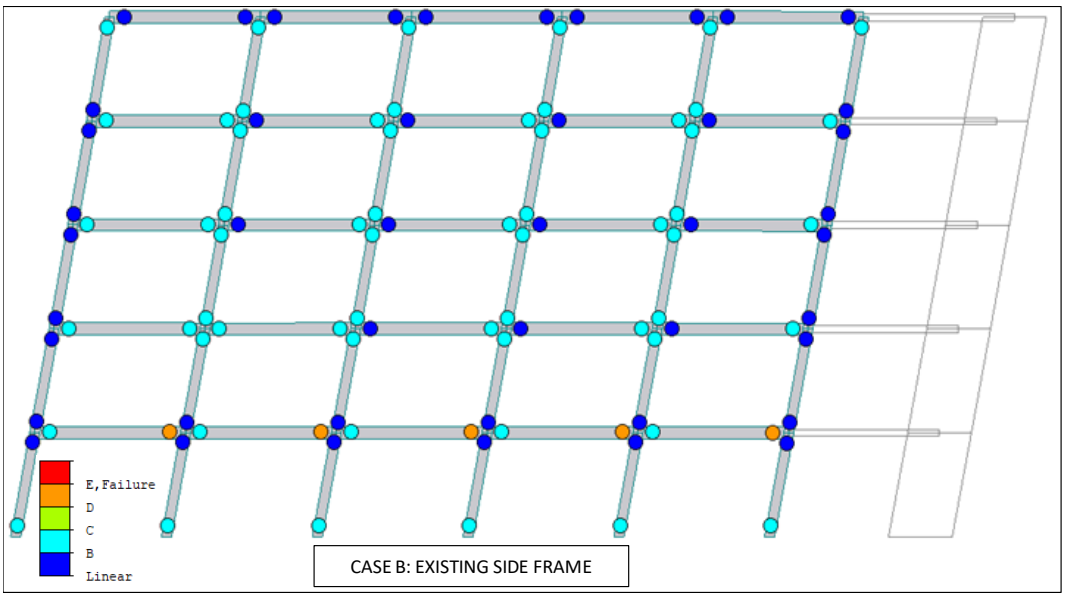

Figure 10: Plastic hinge status result at the step before collapse of frame B in case of Hinged Wall connected at each floor – Model 0 STEP 310 (31 cm) in Figure 9.

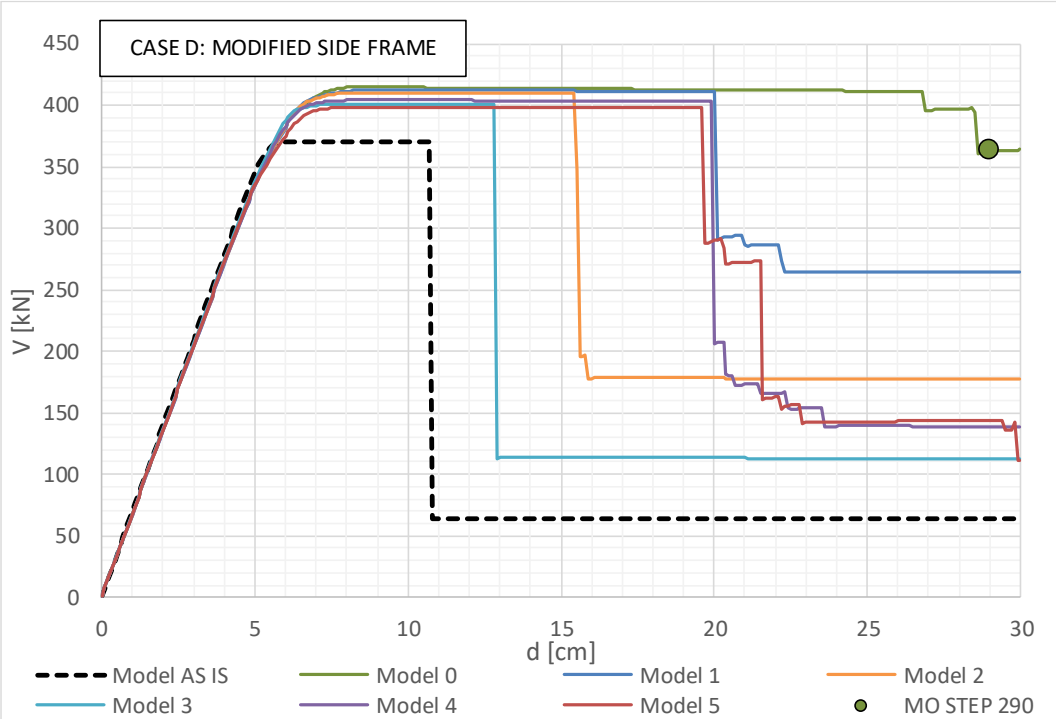

Figure 11: Comparison of pushover results for the frame D different configurations: M0 STEP 290 (29 cm) represents the hinge status result described in Figure 12. Optimal solution: Model 0.

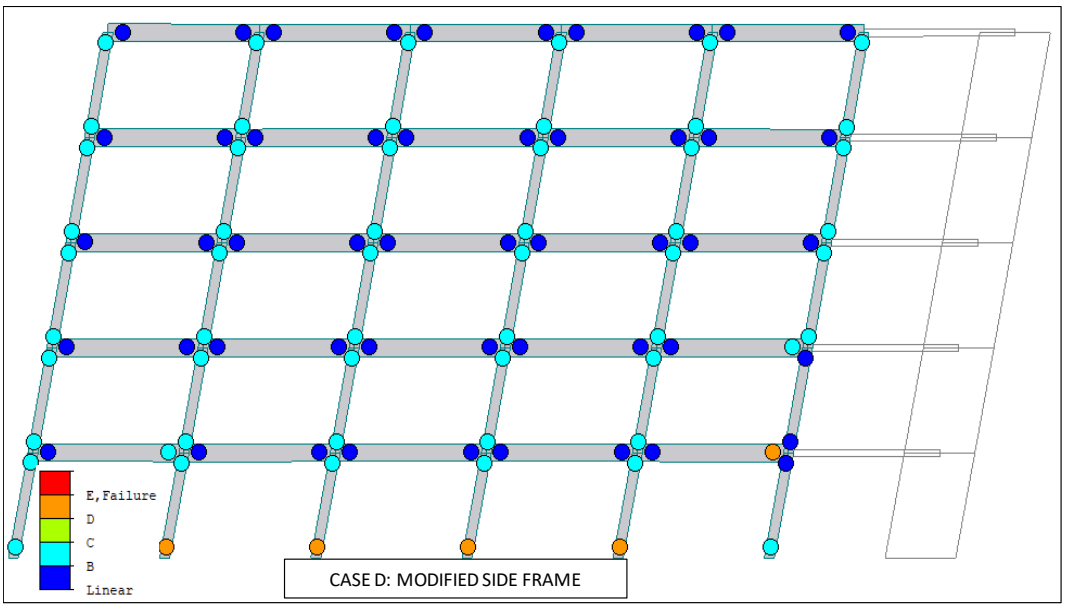

Figure 12: Plastic hinge status result at the step before collapse of frame D in case of Hinged Wall connected at each floor – Model 0 STEP 290 (29 cm) in Figure 11.

4 CONCLUSIONS

The use of hinged walls as retrofit intervention for RC buildings is getting attention worldwide for its low invasiveness and the possibility of carrying out the retrofit intervention form outside. Furthermore, the solution contributes in linearizing the deformation of the building along the height and does not require heavy works to the foundations. In this paper the effectiveness of such solution has been investigated trough nonlinear static analysis of 2D typical RC frames, considering various ratios of the beam-to-column capacity. Some computational aspects were emphasized:

- Modelling dead loads as distributed loads applied to the beams, rather than as point loads lumped at the structure nodes, is crucial to enable the correct estimation of the beam ultimate rotation demand.
- If the moment-axial load interaction is not considered in the RC columns, the effectiveness of the hinged wall application may be overestimated and possible detrimental effects may not be detected.
- The floor nodal ratio, that is the ratio between the beams and columns bending resistance at each floor, seems to be quite relevant in order to identify the collapse mechanism of the existing building and therefore the efficiency of the hinged wall solution.
- Results show that in some cases the adoption of hinged walls may be detrimental of the structural response, if applied in the traditional way (i.e. connecting the wall at each floor). To overcome this drawback, different links configurations have been studied and proposed, in order to identify the optimal solution for each frame typology.

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