# FE MODEL OF RC FRAMES INFILLED WITH RC WALLS FOR SEISMIC RETROFITTING AND NONLINEAR PARAMETRIC STUDY OF DOWELS

E. Georgiou[(1)], N. Kyriakides[(2)], C.Z. Chrysostomou[(3)]

[(1)] *PhD Candidate, Cyprus University of Technology, es.georgiou@edu.cut.ac.cy*
[(2)] *Lecturer, Cyprus University of Technology, nicholas.kyriakides@cut.ac.cy*
[(3)] *Professor, Cyprus University of Technology, c.chrysostomou@cut.ac.cy*

## Abstract

The seismic retrofitting of existing multi-storey multi-bay RC frame buildings by the conversion of selected bays into new RC infilled walls is presented in this paper. This retrofitting method was studied experimentally within the SERFIN project through a series of experiments, which took place at the European Laboratory of Structural Assessment (ELSA) of the Joint Research Centre (JRC) in Ispra. The specimen was a four-storey building tested with the pseudo-dynamic (PsD) method. This was the first time that a full-scale experiment of this type of specimen was performed. The SERFIN structure was designed to represent a typical building in the late 70's early 80's in Cyprus. The frame of the building was designed and detailed for gravity loads only since there were no provisions for earthquake loading at that time. The aim of this experiment was to study the efficiency of the retrofitting method and to examine the required amount of RC wall web reinforcement as well as the connection detail between the wall and the bounding frame. The effectiveness of various connection details between the infill walls and the bounding frame was examined during the experiment. In order to complement the experimental results and to study the interaction between the RC infills and the bounding frame, both in a local and global level, a two-dimensional (2D) finite element (FE) frame model was developed and calibrated using the experimental results. The analysis results of the calibrated FE model showed that the number of dowels connecting the new wall to the bounding frame used in the experiment (with spacing of 100mm), resulted in a monolithic behavior of the RC infilled frame and proved effective. The calibrated model was then used to perform parametric numerical simulation experiments by reducing the number of dowels connecting the new RC infill walls to the existing bounding frame. The variation in the spacing of dowels in the simulated scenarios ranged from 100mm to no dowels. For each scenario, nonlinear response-history analysis was performed. An evaluation of the numerical results and of the FE model simulation of the test specimen is provided along with a comparison between the experimental results and the numerical ones. Top storey displacements, shear-forces and the moments at the base of the frame as well as axial loads of the dowels along the length of the wall at the base interface are presented and conclusions are drawn. The parametric study results provide a basis for the development of a general model for the design of RC infills in existing RC frames, particularly regarding the connection details of the new RC infill walls to the existing bounding frame members.

*Keywords: RC infill walls; finite element model; retrofitting seismic deficient structures; modeling of dowels*

## 1. Introduction

The seismic retrofitting of RC buildings by the conversion of selected bays into new RC infill walls, especially on the perimeter, is a popular, simple, effective and economic strengthening method [1–3]. This method can be applied to increase the strength, stiffness, and ductility of the building. The RC infills as a retrofitting method is commonly applied to guarantee monolithic behavior between the old and the new members to design the new RC walls according to the codes [4, 5]. The monolithic behavior is achieved by the construction of a new thicker web than the beams and the columns of the existing frame panel with the location of the new reinforcement outside the existing members and the details of reinforcement as in a new wall [1]. In this way, the new infill walls are much stronger that what is needed for the strengthening of the structure, and this 'over-strength' causes additional issues like the weak ending of the foundations of the existing buildings [1]. However, the addition of RC infill walls with the same thickness as the frame members that bound the new wall for the seismic strengthening of RC buildings is relatively a new method.
Even though the RC infills is a common retrofitting method and it is extensively applied, it is not addressed quantitatively by the codes, not even by EC8-3 [4]. Specifically, the interaction of new RC infills with the bounding frame, their design and detailing between the new web and the surrounding frame members need to be regulated [1, 4]. The inadequacy of design codes in this respect is due to our poor knowledge of the behavior of walls created by the infilling of a bay of an existing RC frame. Moreover, the experimental research work that has been performed in the last decades on the use of RC infill walls is not adequate and most of the research has mainly targeted large specimens with high resistance [6]. To start filling the gap of knowledge and to study the effectiveness of seismic retrofitting of multi-storey multi-bay RC frame buildings by converting selected bays into new walls through infilling with RC, a full-scale specimen was studied experimentally through PsD test within the project named SERFIN at the European Laboratory of Structural Assessment (ELSA) facility at the Joint Research Centre (JRC), in Ispra. Further details and information about this research work can be found in [7–9]. The results and data from the test of the project SERFIN were used for the simulation and calibration of RC frame infilled with RC walls in FE software to study the behavior of the RC infills within RC frames and to study the interaction between the RC infills and the bounding frame, both in a local and global level.
In this paper, the experimental results are discussed and compared with the numerical ones. Also, the 2D FE frame model that was developed is provided along with the parametric numerical simulation experiments and results that were performed by reducing the number of dowels connecting the new RC infill walls to the existing bounding frame. Top storey displacements, shear-forces and moments at the base of the frame as well as axial loads of the dowels along the length of the wall at the base interface are presented and conclusions are drawn.

## 2. Experimental case study

The subject of the project SERFIN as already mentioned was the retrofitting of multi-storey multi-bay RC frame building by the conversion of selected bays into new infilled RC walls. SERFIN experiment aimed to study the efficiency of the retrofitting method and to examine the amount of the web reinforcement in the walls and the connection details between the new wall and the existing bounding frame. Two parallel planar frames were infilled with RC infills and then they were unidirectional pseudo-dynamically tested. In this section, the SERFIN specimen geometry and design will briefly be described and the experiment test, as well as some of the experimental results, will be presented. The full description and discussion of the experimental campaign, the specimen geometry, details and design of the specimen and experimental results can be found in [3, 7–11].

### 2.1 Specimen geometry and design

The specimen was a full-scale prototype building structure. The center-line length dimension of the specimen was 8.5m, the storey height was 3m, and the total height of the specimen was 12m (excluding the foundation). The two exterior three-bay frames of the prototype structure were infilled with RC walls. These walls were in the central bays of the specimen and they had thickness 0.25m equal to the beams and columns framing them. The infilled frames were named north and south as it is defined in Fig. 1. Hence, the direction towards the reaction wall of the experiment is east and the one away from the reaction wall is west. The results of the south frame of the experiment were simulated and calibrated in the FE software and will be presented as the results of the validated model.

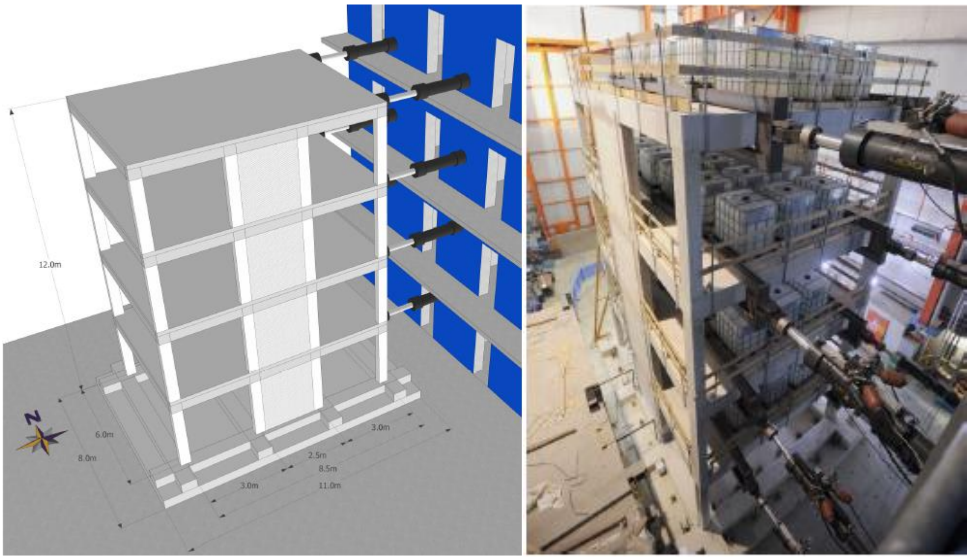



Fig. 1 – Geometry and dimensions of SERFIN specimen with an indication of the cardinal directions. The south frame with infill walls is visible in front. [8]

The SERFIN structure represents a typical construction of the late '70s and beginning of the '80s in Cyprus. At that time, there were no provisions for earthquake loading, so the structures were designed for gravity loads only. For the mock-up design, the provisions of BS8110 was used. The concrete that was used was C20/25 of unit weight 25kN/m³ for both the frame and the walls and modulus of elasticity, E=30GPa. The two infilled frames of the specimen were reinforced with different amount and arrangement of reinforcement, with the North wall being the strongest of the two to facilitate the study of the effect of as many parameters as possible. These details and the full description of the specimen design are given in [3, 7, 9–11].

An additional retrofit was applied to reinforce the edges of the wall on the lap length at the ground floor based on the performance on the ground floor RC infill walls during the experiments. Therefore, three-sided carbon fiber reinforced polymer (CFRP) jackets were applied to reinforce the edges of the wall at the ground floor with a height of 0.6m, since the lapping stop at 0.55m from the base of the column as it is shown in Fig. 2. This retrofit was crucial and constituted part of the proposed retrofit strategy. A lap splice failure would have taken place during the test, which could be detrimental to the whole experiment. Therefore, to safeguard against this type of failure and allow the experiment to be performed successfully, it was decided to reinforce the edges of the wall on the ground floor.

Fig. 2 – Three-sided CFRP jackets to reinforce the edges of the wall on the ground floor. [8]

### 2.2 Experiment test and results

Within the test campaign, two PsD tests and one cyclic test were run. Initially, the accelerogram was scaled to a maximum acceleration of 0.1g for the first test and 0.25g for the second test. The cyclic test was the final test and a history of displacements was imposed on the fourth floor. The south frame results of the 0.25g acceleration test were used for the calibration of the numerical model and their comparison is given in Figs. 4-5.

After the 0.25g acceleration test, the larger level of damage was for the south frame conformed by a crack that opened at the ground beam of the foundation at the base of the wall on both sides and is shown in Fig. 6. Besides, a lap-splice failure due to tensile forces appeared in the outer column on the east side of the south frame as shown in Fig. 6. The presence of the CFRP on the bounding columns of the wall prevented a similar failure, thus allowing the completion of the experiment. The results of the two PsD tests and the final cyclic test are provided and discussed in [8] for both north and south frames.

## 3. FE model and calibration

The simulation of SERFIN full-scale specimen, which reflects correctly the actual behavior of such buildings, was generated in the DIANA FEA tool with a 2D continuum FE model in order to perform numerical experiments and parametric studies. The FE model of the south frame of the SERFIN experiment that was simulated with the same geometry and data of the prototype model is discussed. Also, the experimental results that were validated considering the hysteretic behavior of materials to capture and evaluate the performance of RC infills are described and presented in this chapter.

Suitable elements for the simulation of the RC infills reinforcement and frame members were selected along with material models for concrete and reinforcement, which included hysteretic behavior and strength degradation. In this way, the FE model considers the nonlinear hysteretic behavior of materials during a seismic excitation to capture and evaluate the behavior of RC infills. Moreover, a detailed analysis was obtained considering the nonlinear behavior of the materials at the local level (nonlinear transient analyses were performed). The interaction between the existing bounding frame and the new RC wall was modeled through interface elements, to allow separation of the bounding framing members and the RC wall at their interface when they are in tension. Hence, the dowels that connect the new wall to the existing frame were modeled to take both axial and shear forces, as it is the case in the real structure.

The dead and live loads were applied on the beams as edge pressure load with the same values as the prototype model and the earthquake signal with 0.25g peak acceleration was added as body force for base excitation with earthquake time history function. The nonlinear transient analysis was executed in the FE software. The secant Newton method (quasi-newton), which is an implicit algorithm iterative method was applied, together with the line search method. Convergence tolerance was applied for force and displacements.

A rigid foundation was assumed for the simulation, with pin supports at the base of the frame. The additional weight of the half of the specimen slab and transverse beams was added to the joints of the model through mass point elements. Rayleigh damping coefficients were used with damping of 0.25%.

### 3.1 Elements, mesh and constitutive laws of the FE model

Different elements were selected and applied for all the frame members in DIANA FEA. The full description of the validated FE model simulation is given in [12].

For the concrete members of the frame (columns, beams, and joints) and the infill wall were simulated using the 2D regular plane stress quadrilateral elements (CQ16M, 8 nodes) were used from the DIANA FEA element library. The frame reinforcing bars and the web reinforcement of the RC infills were modeled as reinforcement steel bars and for their mesh, 1D embedded bar reinforcements inside plane stress elements were used, which can carry only axial load. The dowels were modeled in such a way so they can take not only axial but also shear load so, the bond-slip reinforcement with beam elements (BAR LINE, INTERF BEAM) of DIANA FEA were used to model their behavior.

The interface area between the wall and the frame was modeled to capture the tension cut-off behavior between the existing frame and the new infill wall. In this way, the shear strength of dowel reinforcement bars was considered when concrete fails in the FE model. But also, the cohesion and friction between the interfaces are represented at the interface, and at the same time, the opening of the gap when tensile stresses exist between the two interfaces is facilitated in the model. The 2D line interface elements (CL12I) from the DIANA FEA element library were used, having the same thickness as that of the thickness of the plane stress elements representing the wall. The mass of the half weight (312 Tons) of the prototype building was added in the model by using the point mass elements (PT3T) on the 16 joints of the frame.

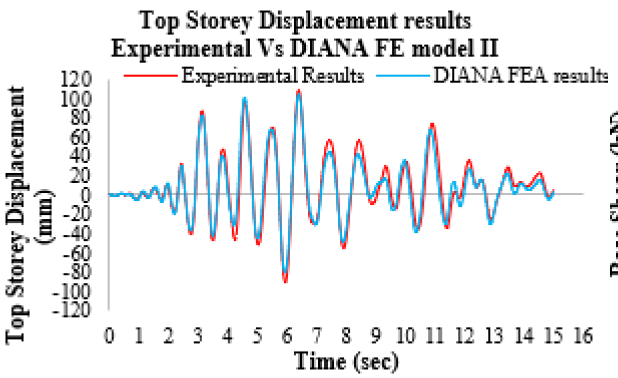
The material models that were applied describe the hysteretic behavior of materials under cyclic loading. More specifically, the selected models simulate the stiffness and strength degradation and the material softening behavior, which causes localization and redistribution of strains in the structure. The full description and definition of the constitutive laws that were applied in the FE model for the concrete members, reinforcement, dowels and interface behavior that were used from the DIANA FEA material library can be found in [12].

### 3.3 Numerical model calibration

The global results of the experiment of the south frame are compared to the DIANA FEA model in Figs. 3-4. Also, the main local results of the experiment that were captured in the real structure are compared with the numerical results and they are illustrated in Fig. 5. The final calibrated model that was developed in DIANA FEA was used to perform the numerical experiments and is discussed in Chapter 4. Indicatively, the comparison of the top-storey displacement of the frame and the base shear force of the frame is compared in Fig. 3. Moreover, the top-storey displacement versus the base shear force is compared in Fig. 4 in order to examine the energy dissipation and the stiffness degradation of the model.

From the FE model's global results, it is clearly shown that the model captures the real structure very well. The peak values are captures in both forces and displacements and the stiffness degradation is captured as well. Furthermore, it is shown that the model captures the frequency of the actual structure.

The main failures that occurred in the SERFIN experiment were a crack opening at the bottom of the column-wall in both sides of the wall and the failure of the east column at the bottom as shown in Fig. 5. From the shear stress distribution in DIANA FEA results of the validated model in Fig. 5, it is shown that the specific failures were captured in the FE model on both sides of the wall. Also, the diagonal strut that was captured on both sides of the wall is shown in Fig. 5. Besides, in Fig. 5, the tensile stress that was reached on the east side of the columns, where the failure of the column occurred at the experiment is apparent in the FE model in Fig. 5.



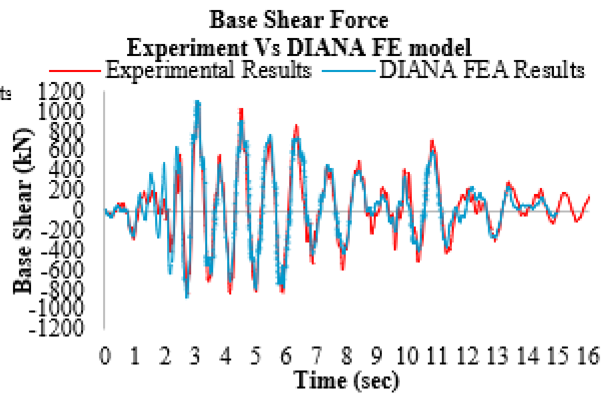




Fig. 3 – Fourth-floor displacements versus time comparison results (left graph), base shear force versus top storey displacement comparison results (right graph).



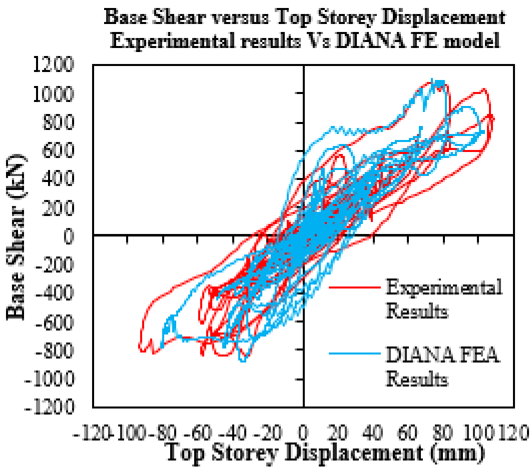



Fig. 4 – Base shear force versus top storey displacement comparison results.

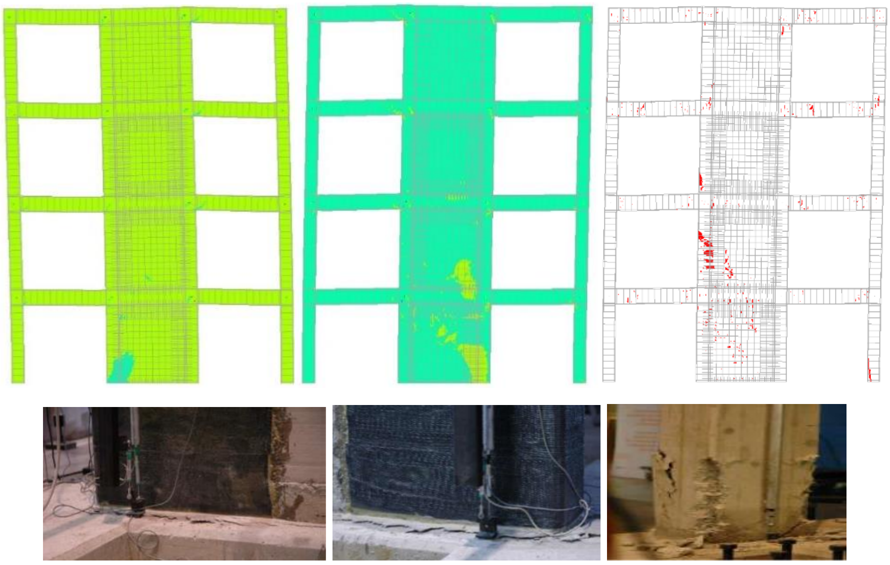


Fig. 5 – Shear stress distribution and diagonal strut in the west and east sides of the wall at 5.8seconds and 6.4 seconds (left and middle Figs. respectively) compared with the crack opened at the bottom of the west and east bounding columns of the south wall, tensile stress of 2.6MPa at 6.4seconds at the east column compared with the tensile lap-splice failure of the ground floor outer column at the east side of the frame (right-hand side of column in the right Fig.). [8]

It is obvious that the FE model is validated, thus is a reliable model of RC infills and its interaction with the surrounding frame through the dowel action at the interface was achieved in the FE model. Eventually, this validated model can be used to study the configurations with a reduced number of dowels to complement the experimental results and to study the interaction between RC infills and bounding frame in the local and global level.

## 4. Numerical parametric study

A parametric study that covers a range between the monolithic behavior (validated model) and that of an infilled frame without dowels, by varying the number of dowels connecting the wall to the bounding frame was performed and the results are presented in this chapter. The validated model had the same number of dowels as in the SERFIN experiment. Another seven different cases of the number of dowels in the model were performed as it is shown in Table 1. The seventh and eighth case scenarios are not shown in Table 1.



For the seventh case scenario two dowels connecting only the beams with the new wall and for the last case scenario (Case 8) there are no dowels in the model, the only connection between the new infill wall and the existing frame members is the cohesion and the friction between the two interfaces. The same analysis procedure and a 0.25g earthquake record were used for all the parametric-study scenarios.

Table 1 – Parametric study scenarios

| Dowels connecting the bounding frame to the wall | Case 1 Validated model | Case 2 14 Dowels | Case 3 10 Dowels | Case 4 6 Dowels | Case 5 4 Dowels | Case 6 2 Dowels |
|---|---|---|---|---|---|---|
| Ground floor columns | 24Y20/100 | 14Y20/140 | 10Y20/250 | 6Y20/460 | 4Y20/760 | 2Y20/2300 |
| Ground floor beams | 20Y20/100 | 14Y20/150 | 10Y20/210 | 6Y20/380 | 4Y20/630 | 2Y20/1900 |
| First floor columns | 24Y18/100 | 14Y18/140 | 10Y18/250 | 6Y18/460 | 4Y18/760 | 2Y18/2300 |
| First floor beams | 20Y18Y100 | 14Y18/150 | 10Y18/210 | 6Y18/380 | 4Y18/630 | 2Y18/1900 |
| Second floor columns | 24Y16/100 | 14Y16/140 | 10Y16/250 | 6Y16/460 | 4Y16/760 | 2Y16/2300 |
| Second floor beams | 20Y16/100 | 14Y16/150 | 10Y16/210 | 6Y16/380 | 4Y16/630 | 2Y16/1900 |
| Third floor columns | 2Y16 | 2Y16 | 2Y16 | 2Y16 | 2Y16 | 2Y16 |
| Third floor beams | 2Y16 | 2Y16 | 2Y16 | 2Y16 | 2Y16 | 2Y16 |

### 4.1 Numerical results of the parametric study

The results from the parametric study of dowels are presented and discussed in this section. Specifically, the top-storey displacements and the base shear forces of the frame from the numerical analysis are illustrated in Fig. 6 and discussed for all the case scenarios. From these results, conclusions are drawn on the effect of the reduction of dowels on the overall stiffness and energy dissipation of the structural system. In each graph in Fig. 6 and Fig. 8, two or three cases are presented for a better comparison between the cases. Also, the initial stiffness of the frames was calculated and is presented in Fig. 7 for all the case scenarios. These results were derived from the initial slope of the graphs in Fig. 6 when the top-storey displacement is 10mm. Moreover, the axial loads of the dowels along the length of the wall at the base interface are presented in Fig. 8 for the first four case scenarios when (i) the top-storey displacements and (ii) the base shear forces of the frames reach their peak in each loading direction. Finally, the moment at the base of the wall (including the bounding columns of the wall) is presented in Figs. 9-10 when the frames experience their maximum base shear forces and top-storey displacements.

The base-shear force of the frame versus the top storey displacement of the frame is given in Fig. 6. It is shown that for the first five case scenarios the total base shear force is about the same. There is a reduction of the base shear force when the dowels are reduced to two in the sixth case. Moreover, the base shear force is reduced in the seventh case scenario where the dowels connecting the bounding columns to the wall were removed. Also, as stated in [13-14], it was shown that after the reduction of the dowels to two there is a change of the elastic characteristics (elastic period) of the frame. From Fig. 6, it is also demonstrated that the highest values of shear forces and top-storey displacements were reached for the first two case scenarios. In Figs. 6 and 7 it is also shown that the energy dissipation and the stiffness of the frame are almost the same for the first two case scenarios. For the third case scenario, it is observed that the top-storey displacement is reduced in the west direction and the energy dissipation of the frame in that direction is shown to be lower than the second case scenario. For the fourth and fifth case scenarios the peak shear and displacement values of the frames are about the same as those of the third. The energy dissipation and the stiffness for the third, fourth and fifth cases are not varying considerably as presented in Figs. 6 and 7. For the sixth and especially for the seventh case scenarios, as already mentioned, the frames take lower base shear forces than the previous case scenarios. In addition, the energy dissipation and the stiffness and the energy dissipation are reduced for the sixth and seventh cases as shown in Figs. 6 and 7. Added to that, one can observe that when the dowels are reduced to two only on beams (seventh case scenario), the stiffness and the energy dissipation of the frame is reduced in comparison to the sixth case scenario. For the eighth case scenario with no dowels, the reduction of the base shear force is significant and the top-storey displacement is very high.



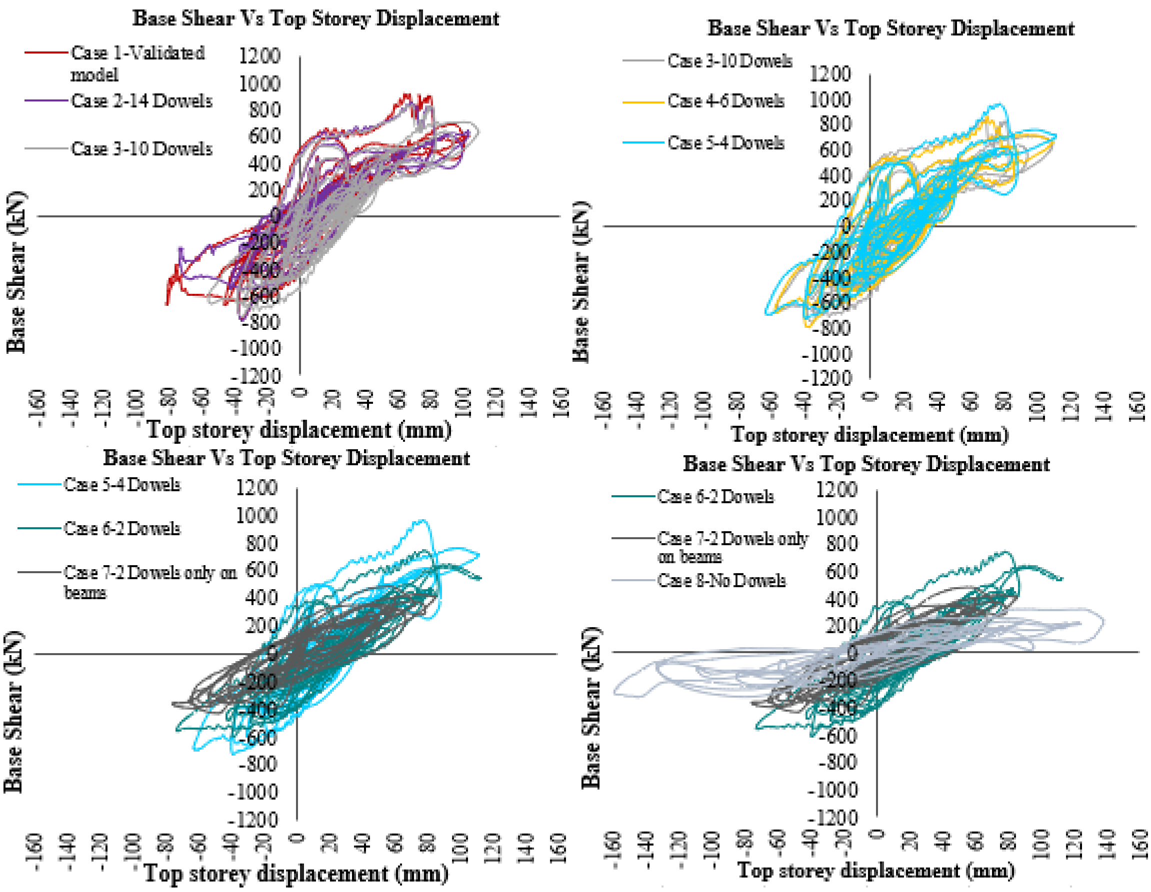



Fig. 6 – Base shear forces versus top-storey displacements for all cases.

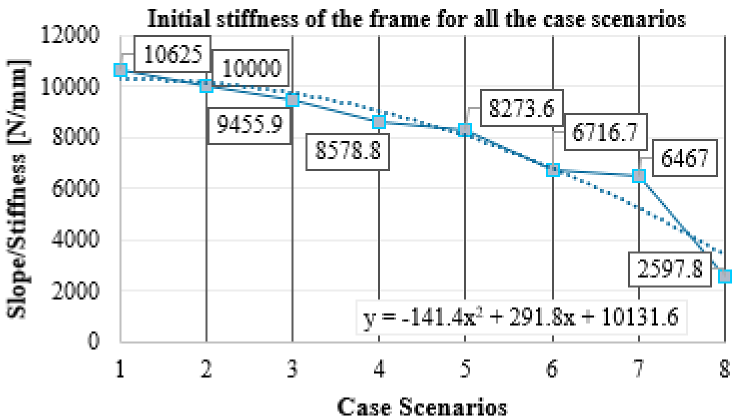



Fig. 7 – Initial stiffness of the frames for all cases.



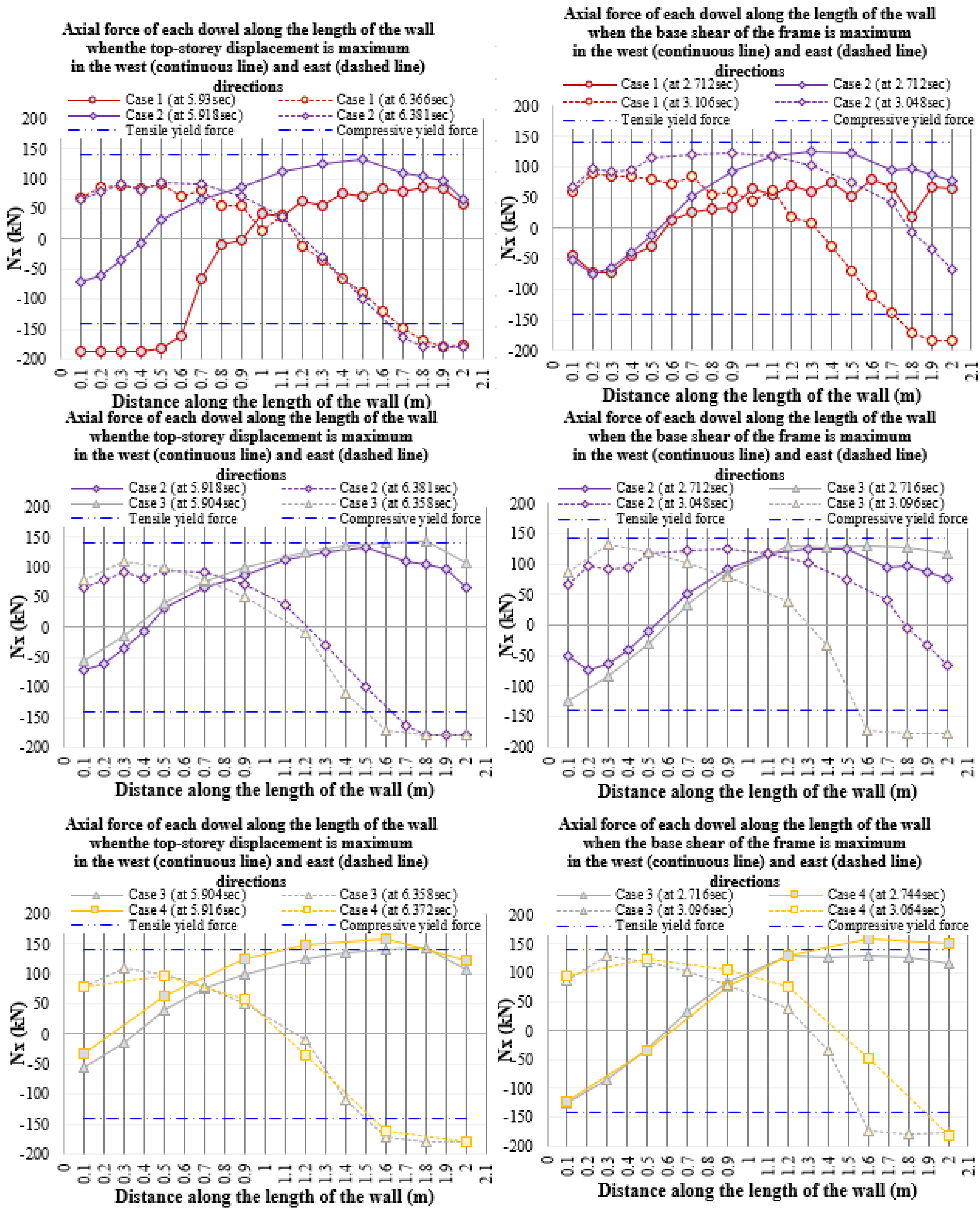
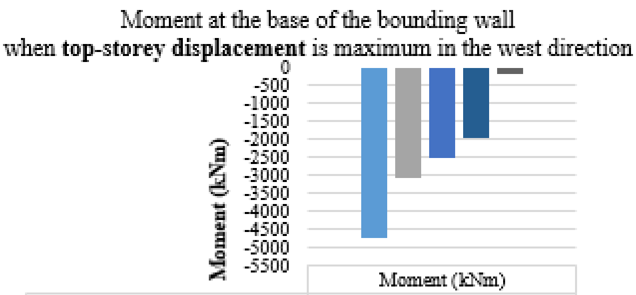


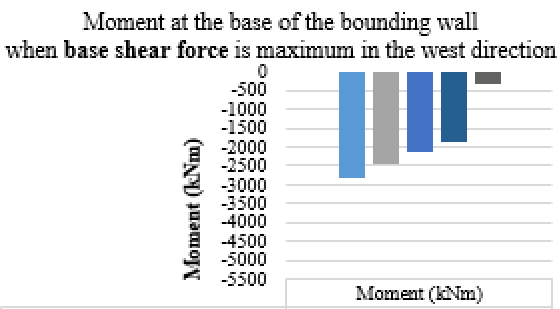

Fig. 8 – Axial force of each dowel along the length of the wall (i) when the top-storey displacement of the frames is maximum in both directions (graphs in the left) and (ii) when the base shear force of the frames is maximum in both directions (graphs in the right) for the first four cases.

In Fig. 8, the axial force of each dowel is presented along the length of the wall. The marker points on the graph corresponds to the actual position of dowels along the length of the wall (0 is the west edge of the wall and 2.1m the east edge of the wall without the bounding columns). As shown in Fig. 8 the maximum values occur at different instances during the analysis for each case. It is shown that the axial forces in the dowels take their lowest values in the first case scenario in comparison with the other cases since the infill wall behaves monolithically with the bounding frame, resulting in an even distribution of the axial force among the dowels. In the second case scenario, the dowels reach higher axial forces in comparison with the first case scenario. Besides, it is shown that the position of the neutral axis in the second case scenario shifts closer to the edges of the wall near the bounding columns, thus more dowels are in tension in both sides of the wall during the peak loading/top displacement in each direction. During the third and fourth case scenarios, some of the dowels reach their yield strength capacity, while with the reduction of the number of dowels, more dowels yield. Therefore, the less the number of dowels the more the number of dowels that yield and the more that are in tension.

The moment demand at the base of the wall, including the bounding columns, as obtained from the axial forces in Fig. 8, when the frame reaches the maximum base shear force and maximum top-storey displacement in each direction. These are illustrated in Figs. 9-10 for the first five case scenarios. In Fig. 9, which refers to the west direction, it is shown that the moment demand at the base of the frames is higher when the frame experiences maximum top-storey displacement for the first three cases. The highest moment at the base of the frame resulted in the first case scenario and for the next cases, the moment demand at the base of the frames is reduced with the reduction of the number of dowels in the west direction. Thus, it can be concluded that the moment demand at the base of the frame is reduced with the reduction of the number of dowels. In Fig. 10, which refers to the east direction, the moment demand is not the highest for the first case scenario, as it was in the west direction, while the highest moment demand is for the second case scenario and then is reduced for the next cases. This is due to the lower axial forces of dowels in the first case in combination with the position of the neutral axis, as shown in Fig. 8, at that time that the moment was calculated.




| | Moment (kNm) |
|---|---|
| Case 1-Validated model (at 5.93sec) | -4722.62 |
| Case 2-14 Dowels (at 5.918sec) | -3047.37 |
| Case 3-10 Dowels (at 5.904sec) | -2526.72 |
| Case 4-6 Dowels (at 5.916sec) | -1958.10 |
| Case 5-4 Dowels (at 5.928sec) | -177.15 |

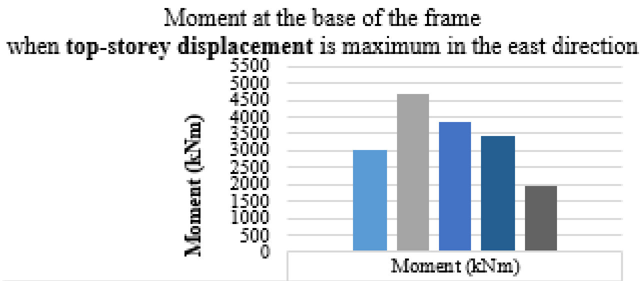



| | Moment (kNm) |
|---|---|
| Case 1-Validated model (at 2.712sec) | -2796.55 |
| Case 2-14 Dowels (at 2.712sec) | -2443.77 |
| Case 3-10 Dowels (at 2.716sec) | -2141.74 |
| Case 4-6 Dowels (at 2.744sec) | -1893.92 |
| Case 5-4 Dowels (at 2.764sec) | -331.20 |

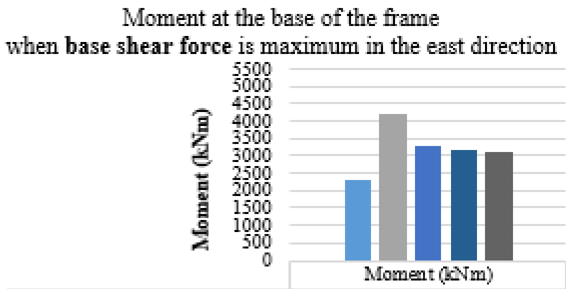
Fig. 9 – Moments at the base of the frames when the top-storey displacement and the base shear of the frames are maximum in the west direction.






| | Moment (kNm) |
|---|---|
| Case 1-Validated model (at 6.366sec) | 3033.14 |
| Case 2-14 Dowels (at 6.381sec) | 4698.26 |
| Case 3-10 Dowels (at 6.358sec) | 3819.18 |
| Case 4-6 Dowels (at 6.372sec) | 3444.52 |
| Case 5-4 Dowels (at 6.388sec) | 1926.18 |




| | Moment (kNm) |
|---|---|
| Case 1-Validated model (at 3.106sec) | 2315.15 |
| Case 2-14 Dowels (at 3.048sec) | 4207.91 |
| Case 3-10 Dowels (at 3.096sec) | 3273.13 |
| Case 4-6 Dowels (at 3.064sec) | 3197.61 |
| Case 5-4 Dowels (at 3.1sec) | 3085.01 |

Fig. 10 – Moments at the base of the frames when the top-storey displacement and the base shear of the frames are maximum in the east direction.

## 5. Conclusions

This study has shown that retrofitting existing RC frames with RC infills can be used to upgrade successfully structures that have been designed for gravity loads only. The numerical simulation of the experimental results of the SERFIN experiment prototype model has shown that the amount of dowel reinforcement used to connect the RC infills with the bounding frame resulted in a monolithic behavior of the infill wall with the bounding frame. The parametric study, which was performed to complement the experimental results and to investigate the effect of the reduction of the number of dowels, used the experiment as a reference point, and reduced the number of dowels in eight steps (cases).

The general trend observed is that the lower the number of dowels at the connection, the lower the maximum base shear force demand as well as the initial stiffness and the energy dissipation of the frame. However, it is shown that these response parameters of the frame do not vary considerably for the first two case scenarios (dowel spacing of 100mm and 150mm) and exhibit an initial reduction in energy dissipation during the third case. For the remaining cases, a reduction in maximum base shear (Fig. 6) and initial stiffness (Fig. 7) occurs progressively.

Regarding the local results of the dowels along the interface of the wall at the foundation, it is observed that during the first case scenario the lowest axial forces in dowels were observed in comparison with the other cases. Higher axial forces in the dowels in cases 3 onwards, lead to the attainment of their yield capacity in tension. Moreover, with the reduction of the number of dowels, the position of the neutral axis is shifting, as expected, towards the edges of the wall in both directions, which indicates that a larger number of dowels are in tension and have reached yielding. Additionally, from the moment results it is shown that the moment demand at the base of the wall at the point of maximum top displacement is reduced with the reduction of the number of dowels. This is an indication that the monolithicity of the connection leads to higher moment demand at the base of the frame.

These results complement the experimental results and show that the number of dowels used in the experimental study can be reduced significantly, making the use of this method more cost-effective. However, further analysis of the numerical results of the other cases will be performed to obtain a better understanding of this structural system that will allow the development of design guidelines.



## 6. Acknowledgments

The research leading to these results has received funding from the European Community's 7th Framework Program [FP7/2007-2012] for access to the EUROPEAN LABORATORY FOR STRUCTURAL ASSESSMENT of the EUROPEAN COMMISSION – JOINT RESEARCH CENTRE under grant agreement no. 227887.

## 7. References

[1] M. N. Fardis, A. Schetakis, and E. Strepelias, "RC buildings retrofitted by converting frame bays into RC walls," *Bull. Earthq. Eng.*, pp. 1–21, 2013, doi: 10.1007/s10518-013-9435-6.

[2] H. Kaplan, S. Yilmaz, N. Cetinkaya, and E. Atimtay, "Seismic strengthening of RC structures with exterior shear walls," *Sadhana*, vol. 36, no. 1, pp. 17–34, 2011.

[3] N. Kyriakides, C. Z. Chrysostomou, P. Kotronis, E. Georgiou, and P. Roussis, "Numerical simulation of the experimental results of a RC frame retrofitted with RC Infill walls," *Earthq. Struct.*, vol. 9, no. 4, pp. 735–752, 2015, doi: 10.12989/eas.2015.9.4.735.

[4] CEN, "Assessment and retrofitting of buildings : Eurocode 8 : design of structures for earthquake reistance : Part 3 : Assessment and retrofitting of buildings / European Committee for Standardization ; editor Cyprus Organisation of Standardisation." Brussels : European Committee for Standardization, c2010, 2010.

[5] KANEPE, "Code for Intervention in Reinforced Concrete Buildings, Earthquake Planning and Protection Organization (OASP)." 2012.

[6] C. Z. Chrysostomou, N. Kyriakides, P. Kotronis, and E. Georgiou, "Derivation of Fragility Curves for RC Frames Retrofitted with RC Infill Walls based on Full-Scale Pseudodynamic Testing Results," in *ECCOMAS Congress 2016*, 2016, p. Paper 16727.

[7] C. Z. Chrysostomou, M. Poljansek, N. Kyriakides, F. Taucer, and F. J. Molina, "Pseudo-dynamic tests on a full-scale four-storey reinforced concrete frame seismically retrofitted with reinforced concrete infilling," *Struct. Eng. Int. J. Int. Assoc. Bridg. Struct. Eng.*, vol. 23, no. 2, pp. 159–166, 2013, doi: 10.2749/101686613X13439149156831.

[8] M. Poljansek *et al.*, "Seismic Retrofitting of RC Frames with RC Infilling (SERFIN Project)," Publications Office of the European Union, Luxembourg, 2014.

[9] C. Z. Chrysostomou, N. Kyriakides, M. Poljansek, F. Taucer, and F. J. Molina, "RC infilling of existing RC structures for seismic retrofitting," in *Geotechnical, Geological and Earthquake Engineering*, 2014.

[10] C. Z. Chrysostomou and N. Kyriakides, "Pseudo-Dynamic Tests on a Full-scale 4-storey RC Frame Seismically Retrofitted with RC Infilling," 2013.

[11] C. Z. Chrysostomou, Kyriakides Nicholas, Kotronis Panagiotis, Georgiou Elpida, "RC Infilling of Existing RC Structures for Seismic Retrofitting," in *Second European Conferece on Earthquake Engineering and Seismology*, 2014.

[12] E. S. Georgiou, N. C. Kyriakides, C. Z. Chrysostomou, P. Kotronis, and C. A. Filippou, "Numerical Simulation of the Experimental Results of the Seismic Strengthening of Existing Structures," in *16th European Conference on Earthquake Engineering*, 2018, p. Paper 11029.

[13] E. Georgiou, C. Chrysostomou, and N. Kyriakides, "Nonlinear Numerical Parametric Study of Dowels for the Seismic Strengthening of Rc Frames With Rc Infill Walls," no. June, pp. 3100–3113, 2019, doi: 10.7712/120119.7135.19654.

[14] E. Georgiou, C. Z. Chrysostomou, and N. Kyriakides, "Ενίσχυση Φορέων ΟΣ με Τοιχοπληρώσεις ΟΣ Retrofitting of RC frames with RC infill walls," *4th Hell. Natl. Conf. Earthq. Eng. Tech. Seismol.*, pp. 1–11, 2019.