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INFLUENCE OF NON-STRUCTURAL INFILL WALL ELEMENTS ON THE SEISMIC RESPONSE AND RETROFIT OF STEEL STRUCTURES

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Abstract

This work provides a methodology for evaluating the influence of non-structural infill wall elements and quantifying the interrelation between them and structural damage analytically evaluated under strong ground motions. For this reason, nonlinear dynamic analyses for a set of five steel frame structures were carried out taking into consideration all essential material characteristics as well as four different infill wall topologies. Initially, a set of spectrum compatible artificial accelerograms were composed, and nonlinear dynamic analyses have been carried out to evaluate the seismic response of each structure. From several structural response quantities, the overall structural damage index (OSDI) of Park/Ang ($OSDI_{PA}$) has been selected to represent the structural response. The work focuses on damage index of Park/Ang (DI_{PA}) in both its localized form as well as the global damage manifestation. The steel frame models were designed in compliance with EC3 and EC8 Eurocode requirements for steel and antiseismic structures, respectively. During the nonlinear dynamic analyses carried out to evaluate the structure's seismic response, utilizing the IDARC computer program, the $OSDI_{PA}$ response parameters were calculated as simple, yet efficient and widely accepted ways to represent seismic damage. This work focus on quantifying the interdependency between the topology of infill walls in a steel structure and its seismic response. As the numerical results have shown, infill walls proved to have a very positive contribution to the structure's seismic response giving an average of 47% reduction of the maximum recorded $OSDI_{PA}$ as well as a 72-81% decrease in the mean $OSDI_{PA}$ values between the bare frame structure and its infill wall reinforced counterparts. Similar results were observed on a localized basis when investigating the mode and structural damage accumulation on a level to level basis where reductions in the range of 68-82% of the mean values for the ground floor (Level 1) and almost eliminate the seismic damage that is observed on the bare frame's Level 6. Finally, a comparative study has been performed to quantify the influence of non-structural infill wall topology on the analytically observed seismic induced structural damage. In that effect, the different damage distribution characteristics have been studied revealing the importance of non-structural elements in a building's response to such conditions.

Keywords: Seismic Parameters; Damage Indices; Frame Structures; Infill Walls; Damage Distribution

1. Introduction

The frame structure is the predominant form of construction for mid-rise buildings due to its inherent advantages in terms of its relatively low mass over structural height ratio and straight forward response properties under horizontal excitation. Steel frame structures are a rather common type of building type extensively utilized where the speed of construction is of the essence. In most cases infill walls, depending on the architectural considerations, cover the whole or part of one or more of the steel sub-frames. Due to the very nature of steel structures and their ability to adapt its design conditions at a post construction stage to cater for different types of use, with different internal space arrangements, designers select to provide maximum transformability to the respective occupants by arranging the structural system in order to maximize free areas. The inclusion of infill walls at that stage can either be as part of the structural system or not, nevertheless in most cases to maintain the adaptable nature of a steel structure clients and designers call for steel structures to be designed and constructed based on the assumption that all horizontal actions be accommodated by a frame resisting design.

This enhances the modularity of internal spaces and introduces simplifications to the structural complexity of the solution in terms of analysis and design rendering it an appealing process. Paulay and Priestley [1] in their theory regarding the seismic behaviour of masonry infilled frames called for alteration, the existence of such infill elements bring, to the structural system: resulting in improved overall lateral load capacity. More contemporary work [2] has utilised the bracing of moment resisting steel frames as a means of improving the existing seismic response characteristics with great benefit, and similar results have been recorded from site surveys to the affected building stock after strong motion events. Identifying the positive effects of such an interaction between structural and "non-structural" elements the Federal Emergency Management Agency (FEMA) prepared FEMA 273 [3] in the NEHRP Guidelines for the Seismic Rehabilitation of Buildings provisions, dictating that concrete frames with infill walls must be constructed in such a way as to ensure infill element and frame interact under design loads.

Results of analytical and experimental analysis acknowledged that infill walls contribute to the modal response of structure [4] in terms of the overall building structural rigidity, the structure’s natural period and damping coefficient. Similarly, in steel frame structures, much effort has been spent on research into the contribution of infill walls on seismic characteristics; leading to the identification of several important properties of infill walls in terms of their contribution to overall seismic behaviour in the case of structures with bare ground floor [5]. This work provides a methodology to quantify the interrelation between the inclusion of "non-structural" infill wall elements on a bare frame structure and its effect structural response and recorded damage during strong seismic events.

2. Methodology

This work identifies and quantifies the effects of non-structure infill wall elements in the overall seismic behaviour of a steel frame structures outlining their potential significance. To assess the aforementioned a simplified analytical model of a typical commercial steel frame 10 storey building has been constructed and 4 different non-structural infill wall topologies have been realised. All structural elements and connections were designed in such way as to be in compliance with the relevant recent Euro codes for steel and antiseismic structures for steel moment resisting frame buildings, EC3 [6] and EC8 [7] respectively, hence representing a typical contemporary steel structure. A total of 225 artificial accelerograms compatible with the Greek Antiseismic Code [8] have been employed, and nonlinear dynamic analysis has been carried out to evaluate the structural response for the different models involved. The overall structural damage index of Park/Ang ($OSDI_{PA}$) and the Level Damage Index of Park/Ang (LDI_{PA}) have been selected to characterize both localized and global damage status.

2.1 Synthetic accelerograms

The seismic excitations used for the dynamic analyses in this study are based on artificial accelerograms. The reason for choosing this approach was dictated by the need for a sufficiently large statistically robust database. In order to avoid the limitations regarding statistical coherence and wide spread of recorded structural damage, many artificial accelerograms were created. Strong motion acceleration time-histories have been created that match the desired peak ground accelerations. In this case a methodology of specifying a smooth design response spectrum on which the created artificial strong motion events will be based was used. For the creation of such artificial accelerograms, the program SIMQKE [9] has been used.

All the above were based on the assumption of category B subsoil, that calls for deep deposits of medium dense sand, or over-consolidated clay at least 70 m thick, as described in Eurocode 8 and the Greek Antiseismic Code [8] with the use of a differentiated choice of seismic parameters 225 artificial accelerograms have been created, all compatible with the relevant code response spectra. Those parameters where the peak ground acceleration (PGA), the total duration (TD) of the seismic event (with TD values of 20 s, 30 s and 40 s) and the design spectra acceleration (α) for all three Greek seismic regions (nominal α equal to 0.16g, 0.24g and 0.36g) as shown in Table 1.

Table 1 – Synthetic accelerogram composition Table

Target Response Spectrum (g)	Total Duration (s)	Target PGA (g)				
Greek Seismic Zones I/II/III 0.16g/0.24g/0.36g	20	0.12	0.15	0.20	0.25	0.30
	30	0.17	0.20	0.25	0.30	0.35
	40	0.27	0.30	0.35	0.40	0.45

2.2 Damage indices

As explained previously, attention is focused on damage indicators that consolidate all member damage into one single value that can easily and accurately be used for the statistical exploration of the interrelation with the single-value seismic parameters in question. Thus, in the OSDI model of Park/Ang [10] the global damage is obtained as a weighted average of the local damage at the ends of each beam or column element. The local damage index is given in Eq. (1).

$$DI_L = \frac{\theta_m - \theta_r}{\theta_u - \theta_r} + \frac{\beta}{M_y \theta_u} E_T \tag{1}$$

where, DI_L is the local damage index; θ_m the maximum rotation attained during the load history; θ_u the ultimate rotation capacity of the section; θ_r the recoverable rotation at unloading; β is a strength degrading parameter; M_y the yield moment of the section; and E_T the dissipated hysteretic energy. The Park/Ang damage index is a linear combination of the maximum ductility and the hysteretic energy dissipation demand imposed by the earthquake on the structure. The global DI of Park/Ang is presented in Eq. (2).

$$OSDI_{PA} = \frac{\sum_{i=1}^{n} DI_L E_i}{\sum_{i=1}^{n} E_i} \tag{2}$$

where, $OSDI_{PA}$ is the global damage index of Park/Ang; DI_L the local damage index of Park/Ang; E_i the energy dissipated at the location i; and n the number of locations at which the local damage is computed. In the same context, the localised form of $OSDI_{PA}$ has been evaluated, as the sum of the recorded DI_L concentrated at each respective level, providing a local damage index relevant to each separate level as shown in Eq. (3).

$$LDI_{PA} = \frac{\sum_{i=1}^{n} DI_{LL} E_{iL}}{\sum_{i=1}^{n} E_{iL}} \tag{3}$$

where, LDI_{PA} is the level structural damage index of Park/Ang; DI_{LL} the local damage index of Park/Ang for a particular level; E_{iL} the energy dissipated at location i of the level in question; and n the number of locations at which the local damage is computed.

3. Numerical Model

The geometry, layout and the structural elements profiles of the 5 different 10 storey building structural models have been selected to represent a wide variety of real-world scenarios. As such Frame 0 has been chosen as being the bare frame one and Frames 1-4 have been selected with 2 outer bays bearing infill; central bay bearing infill; same as before but with no infill at ground level respectively (see Fig.1).

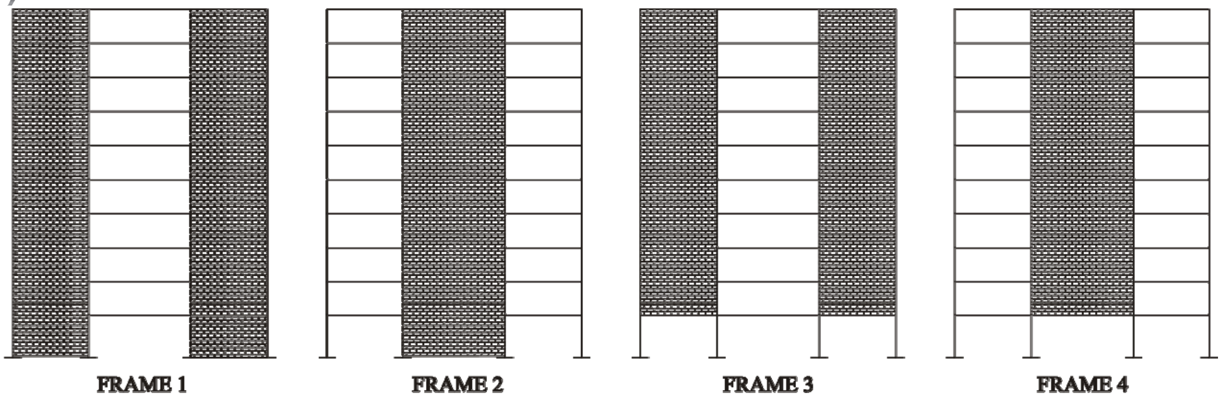

Fig. 1 – Infill wall topologies

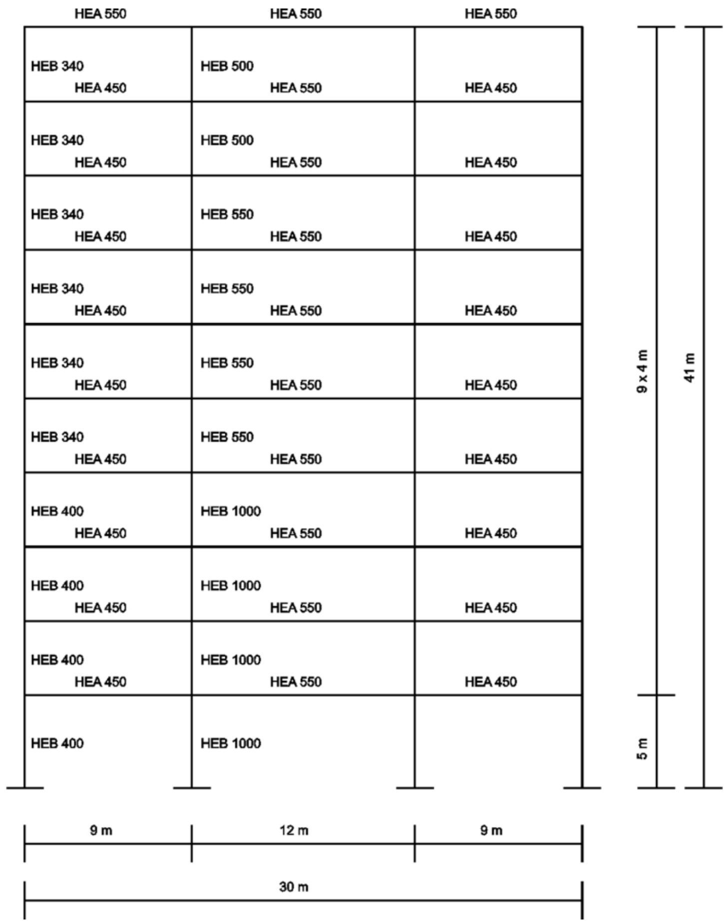

Fig. 2 – Bare steel frame

Structural detailing was completed by implementing the requirements of both EC3 [6] and the current Greek antiseismic code [8] for steel anti-seismic structures. The slabs' thickness has been designed based on the assumption of a building of importance category 2 (common buildings), low ductility requirements, type B subsoil (deep deposits of medium dense sand or over-consolidated clay at least 70 m thick) belonging to a seismic zone I (a = 0.16g) according to the Greek antiseismic code. The loads safety factors, as well as load combinations, have been chosen in accordance with Eurocode 1, 3 and the Greek antiseismic code requirements.

The load values used for structural design constitute of an imposed load of 5 kN/m^2, a snow load of 0.75 kN/m^2 for the roof, wind action according to EC1, concrete slab self-weight assumed to come from a C20/25 concrete slab with a depth of 200 mm while the infill walls were considered as loads coming from a single non-load bearing infill element of 140 mm thickness. In addition to the above, the eccentricity of structural elements from verticality has been accounted for as per the nominal values pertaining to the relevant construction codes for the design of such structural frame. The frame design parameters, as well as element dimensions are presented in Fig. 2 and the material utilised was S355.

4. Analysis

The computer program IDARC-2D [12] has been utilised. A three-parameter Park model was elected to represent the hysteretic behaviour of beams and columns at both ends of each member. This hysteretic model incorporates stiffness degradation, strength deterioration, slip-lock and a trilinear monotonic envelope. Experimental results of cyclic force-deformation characteristics of typical components of the studied structure specify the parameter values of the above degrading parameters with the use of the nominal parameter for stiffness degradation. IDARC utilises the Newmark-β method of numerical integration and Newton/Raphson's iteration method for every time step. A bi-linear elasto-plastic model with 5% offset yield strength has been selected to represent the steel elements' behaviour. The steel material has been modeled as a von Mises material with isotropic hardening. Plastic strains were included with the bilinear elastic–plastic stress–strain curve with 5% linear strain hardening used to simulate the steel material as per Figure 3 while the ultimate deformation (curvature) for members was specified as the lowest of either the maximum strain at fracture divided by the neutral axis, or the maximum plastic moment and a post-yield hardening capacity of 0.05. The infill walls have been incorporated in the form of diagonal compression struts in the respective sub-frames. The smooth hysteretic model that was also used for the infill panels include the effects of stiffness degradation, strength deterioration, and pinching. The development of the present hysteretic model is based on the non-linear Bouc-Wen model [12]. Finally, the structural response of the building for the artificial accelerograms under investigation was based on the extraction of the overall structural damage index of Park and Ang in light of its ability to consolidate recorded damages in one numerical value. For reasons of non-linear dynamic analysis execution the stress-strain model selected for the infill wall element in compression is presented in Figure 4 and constitutes of a parabolic part up to the maximum permissible stress f'_m and then reduced at a lower point where it remains constant.

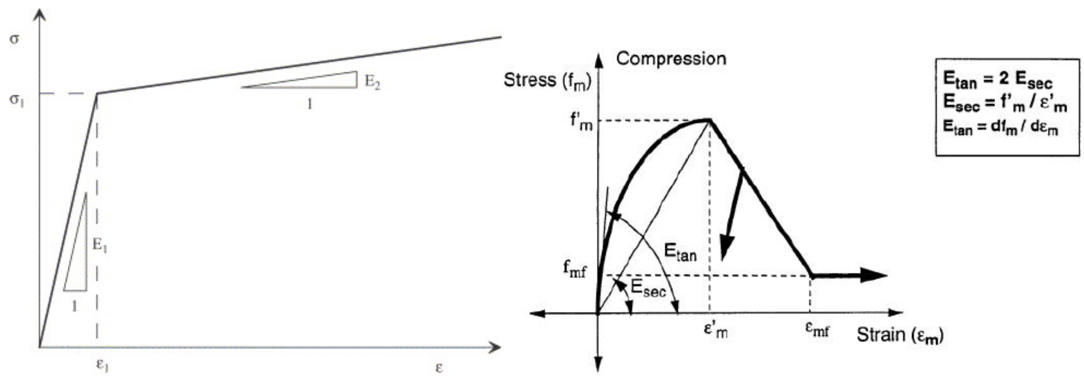

Fig. 3 & 4 – Structural steel & Infill wall element Stress-Strain diagram

The infill wall element's diagonal struts are considered inactive when in tension but the combined action of the two diagonals provides the necessary resistance from both directions. The relationship between the horizontal force-displacement-model of the diagonal strut system is shown in Fig. 5 while a mild hysteretic Bouc-Wen behaviour model shown in Fig. 6 has been utilized.

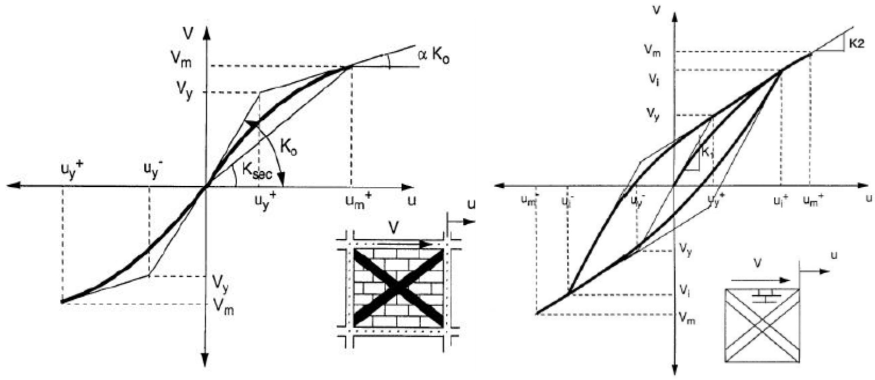

Fig.5 & 6 - Relationship between horizontal force and displacement in infill wall elements & Bouc-Wen model for the mild hysteretic behaviour of infill wall elements

For the calculation of the hysteretic response of the infill wall sections the Saneinejad and Hobbs [13] allowable compression diagonal strut element calculation Eq. (5) have been utilised for the permissible compression f_Q with values of $f_c = 0.6 \cdot \varphi \cdot f_m$ and $\varphi = 0.65$.

$$f_Q = f_C \left[1 - \left(\frac{L_{eff}}{40t} \right)^2 \right] \tag{5}$$

5. RESULTS

Upon completion of the results the influence of the non-structural infill wall elements becomes apparent with an overall reduction of the average $OSDI_{PA}$ in those frames ranging from 72 to 80% while values ranging from 26 to 4% have been observed in terms of the overall 95% confidence upper and lower mean values between the bare frame and its non-structurally infilled counterparts. Details for the above are presented in Table 2 and explained throughout chapter 6. The overall distribution of $OSDI_{PA}$ values between the different frames is presented in Fig. 7.

Table 2 – $OSDI_{PA}$ mean and 95% confidence mean reduction values

% Reduction in $OSDI_{PA}$ between Bare Frame (Frame 0)		
Frame	Mean	95% confidence Mean
1	80	26
2	79	22
3	72	4
4	73	4

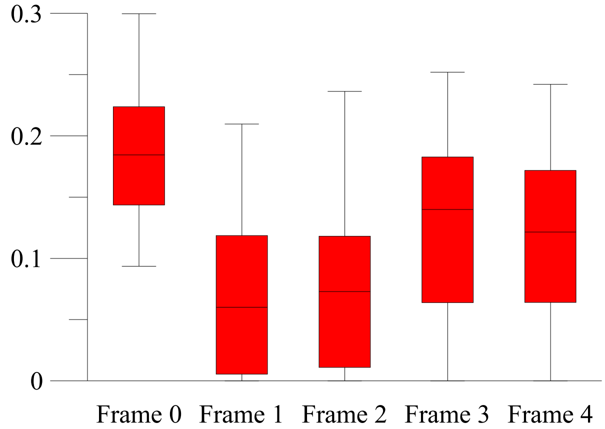

Fig. 7 – $OSDI_{PA}$ values and distribution between frames

A detailed analysis of the results has been undertaken to identify the distribution of LDI_{PA} values for the frames in question in order to assess the post-seismic structural behaviour. An interesting feature of Frame 0 became apparent with nature and values recorded for Level 6. In Fig. 8 the increased concentration of high LDI_{PA} values demonstrate the susceptibility of this design to the given strong motion conditions.

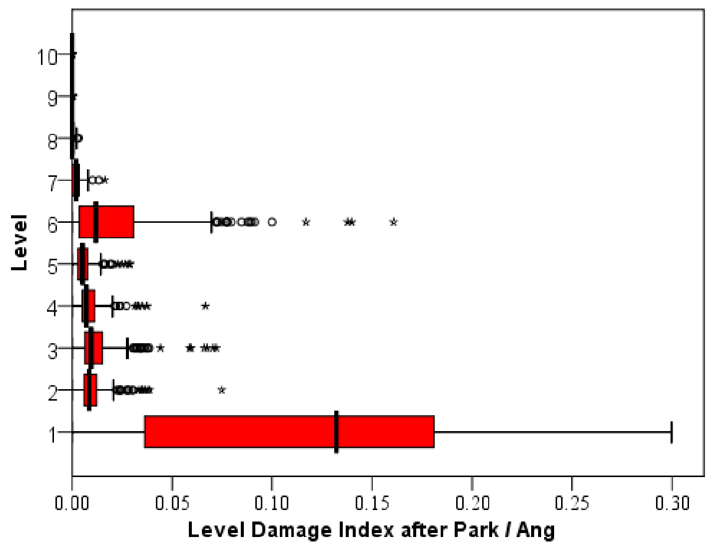

Fig. 8 – LDI_{PA} distribution between different levels of Frame 0

It is, therefore, important to identify the behaviour of the infilled frames in terms only of their overall behaviour but also in regard to their local damage distribution as per the above. Fig. 9 interposes the available results in and provides insight as to the effect that the non-structural elements have on the behaviour of the steel frame structure.

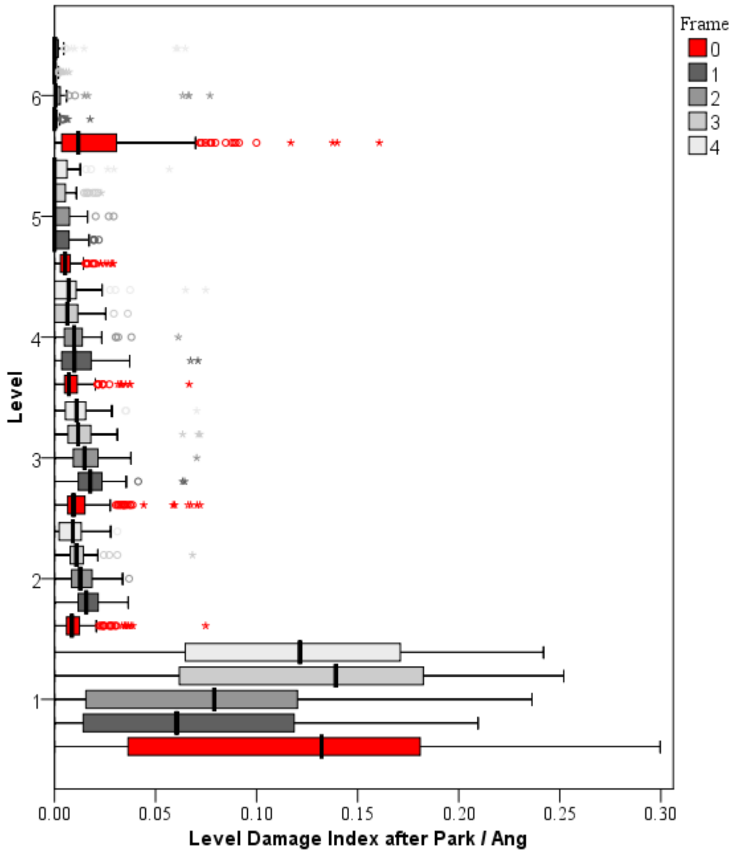

Fig. 9 – LDI_{PA} distribution between different levels for all frames

Evidence of significant behavioural influence can be seen and in regards to the non-structural nature of the infills. As such, a notional soft storey effect is starting to develop but with a considerable redistribution of damage throughout the full height of the structure that explains the overall reduction of the $OSDI_{PA}$ values. The results refer to non-zero $OSDI_{PA}$ cases to avoid skewing the results towards zero. Table 3 provides the mean LDI_{PA} values per floor for each frame and Table 4 the respective reduction observed in (%) of the bare frame values.

Table 3 – Mean LDI_{PA} values distribution for all instances with non-zero $OSDI_{PA}$

Level / Frame		1	2	3	4	5	6	7	8	9&10
Mean LDI_{PA}	0	0.1186	0.0105	0.0133	0.0094	0.0062	0.0233	0.0023	0.0005	No Significant values
	1	0.0215	0.0048	0.0058	0.0038	0.0013	0.0004	---	---	
	2	0.0239	0.0038	0.0048	0.0033	0.0014	0.0018	0.0000		
	3	0.0394	0.0037	0.0046	0.0025	0.0012	0.0002	---		
	4	0.0377	0.0029	0.0038	0.0030	0.0014	0.0013	0.0000		

Table 4 – Mean LDI_{PA} (%) reduction distribution for all instances with non-zero $OSDI_{PA}$

Level / Frame		1	2	3	4	5	6	7-10
Level Damage Index reduction from Frame 0 (%)	1	82	54	56	60	79	98	No Values of significance
	2	80	64	64	65	77	92	
	3	67	65	65	73	81	99	
	4	68	72	71	68	77	94	

As such, a dramatic influence is observed in the infilled frame structures with a reduction of anywhere from 54 to 99% on the recorded level damage index. To further examine those results in terms of robustness the use of the upper and lower 95% confidence boundaries of the mean values has been utilised. That way an evaluation of not only the mean but the general behaviour of the structure for the selected case studies could be carried out. A simplified method of comparing the distances between the upper and lower confidence limits has been utilised resulting in an assessment of the proximity of such values as an indirect measurement of different topologies structural response uniformity. Fig. 10 to 15 present the results while Tables 5 and 6 provide an analysis of those values and Table 7 the post process results that identified said robustness.

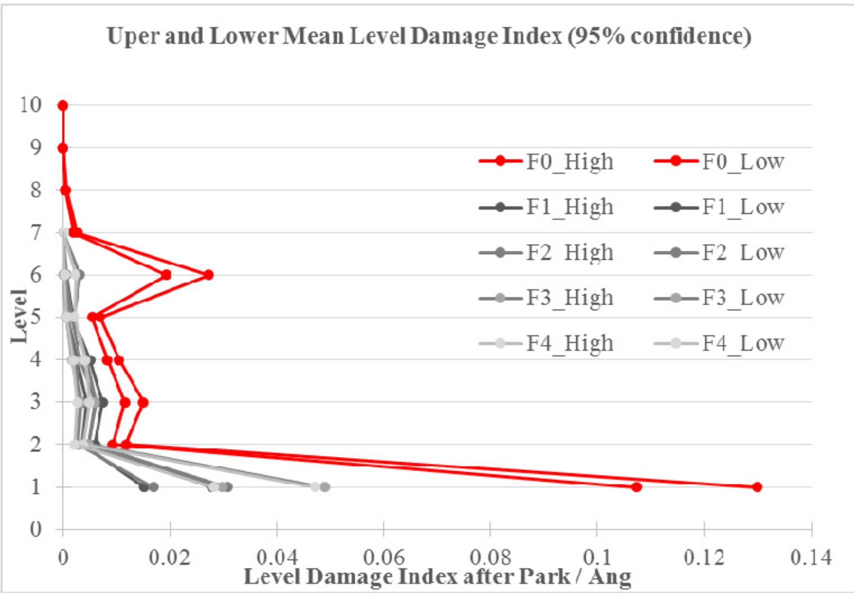

Fig. 10 – 95% confidence mean LDI_{PA} distribution between levels for all frames

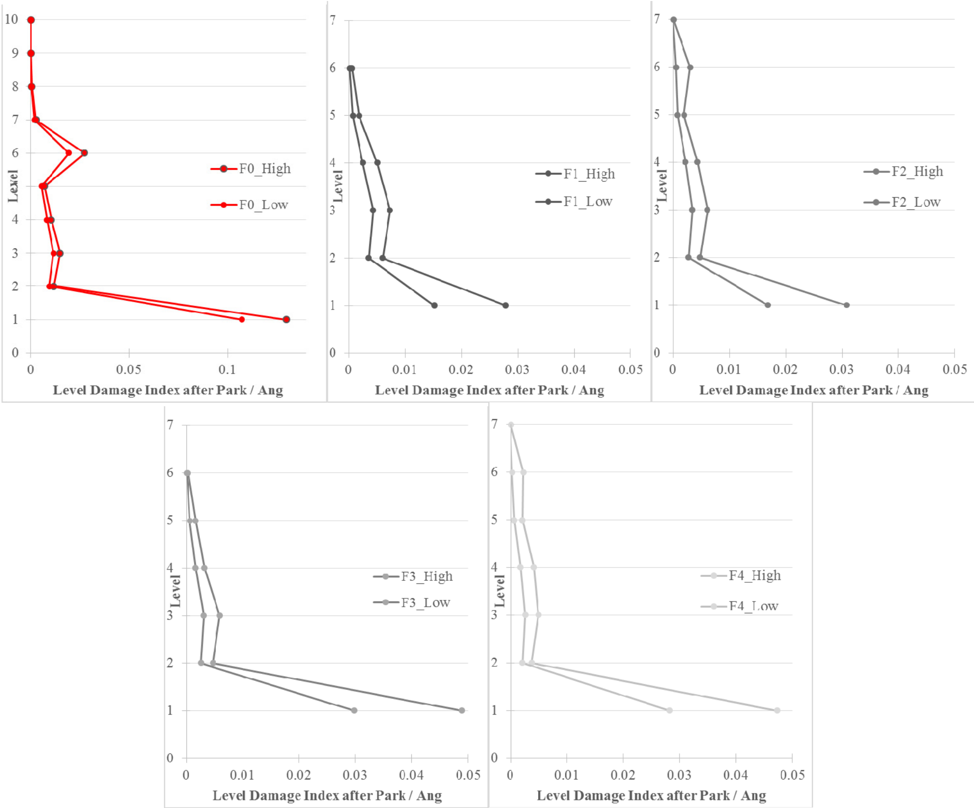

Fig. 11 to 15 – 95% confidence mean LDI_{PA} distribution between levels for each frame

Table 5 – Mean LDI_{PA} values for Frame 0

Level		1	2	3	4	5	6	7	8
Mean LDI_{PA}		0.1186	0.0105	0.0133	0.0094	0.0062	0.0233	0.0023	0.0005
95% Mean Confidence	Lower	0.1073	0.0093	0.0116	0.0083	0.0054	0.0194	0.0019	0.0004
	Upper	0.1299	0.0118	0.0150	0.0105	0.0069	0.0273	0.0026	0.0006

Table 6 – 95% confidence mean LDI_{PA} values for Frames 1 to 4

Level	Frame 1		Frame 2	
	95% Mean Confidence		95% Mean Confidence	
	Lower	Upper	Lower	Upper
7	---	---	0.0000	0.0001
6	0.0002	0.0006	0.0005	0.0031
5	0.0007	0.0018	0.0008	0.0019
4	0.0025	0.0050	0.0022	0.0043
3	0.0043	0.0074	0.0035	0.0061
2	0.0036	0.0060	0.0028	0.0048
1	0.0152	0.0279	0.0169	0.0308
Level	Frame 3		Frame 4	
	95% Mean Confidence		95% Mean Confidence	
	Lower	Upper	Lower	Upper
7	---	---	0.0000	0.0001
6	0.0001	0.0003	0.0003	0.0023
5	0.0006	0.0017	0.0007	0.0022
4	0.0017	0.0033	0.0018	0.0041
3	0.0032	0.0060	0.0027	0.0050
2	0.0026	0.0047	0.0021	0.0037
1	0.0299	0.0490	0.0282	0.0472

Table 7 – 95% confidence mean LDI_{PA} (%) reduction distribution for all instances with non-zero $OSDI_{PA}$ against the bare frame

Frame \ Level		1	2	3	4	5	6	7-10
Level Damage Index reduction from Frame 0 (%)	1	44	2	11	-14	27	95	No Values of significance
	2	38	17	22	6	23	68	
	3	16	15	17	28	30	97	
	4	16	35	32	-5	3	75	

6. Conclusion

This work has revolved around the influence of non-structural infill wall elements on the seismic response and retrofit of steel structures. This work quantified the influence of non-structural infill wall elements on a bare frame structure in both a global and local domain for a set of discretely different non-structural element topologies. A set of 225 artificial accelerograms has been composed and used in nonlinear dynamic analyses providing the structural response of the structure. The structural damage results were quantified with the help of the overall structure damage index (OSDI) of Park/Ang (DI_{PA}) as well as its Local equivalent (LDI_{PA}) while statistical analyses were implemented to ensure the uniformity of results and the applicability of such findings in a generalised framework.

All presented numerical results showed a significant reduction in the overall damage indices for the infill wall topologies selected against the bare frame structure demonstrating the positive effect of non-structural elements under seismic conditions. In summation, the improvement in seismic structural behaviour recorded with infill walls utilization, in comparison to the bare frame structure, manifested with an overall reduction averaging 47% in the maximum recorded $OSDI_{PA}$ as well as an 72-81% decrease in the mean $OSDI_{PA}$ values between bare frame structure and its infill wall reinforced counterparts. While similar results were observed on a localized basis when investigating the mode and structural damage accumulation on a level to level with reductions in the range of 68-82% of the mean values for the ground floor (Level 1) and almost an elimination of the seismic damage that is observed on the bare frame's Level 6 the nature of those reductions allows confidence about their ability to be generalised to similar structures. The statistical analysis of the results utilising the 95% confidence upper and Lower mean boundaries showed the robustness of the recorded values demonstrating smaller confidence intervals than the bare frame benchmark frame structure.

The above further strengthens the derived conclusion regarding the nature of non-structural elements and the effect they can have in the seismic structural response. It is, therefore, necessary to ensure compliance of the structural response characteristics to the desired design provisions when such elements are added to new or existing structures to avoid adverse effects as well as consider their applicability as means of retrofit in cases where a mild structural rehabilitation approach could be of use or interest. Based on the above the need for further development of the necessary engineering framework, addressing structural design optimization with the use of non-structural elements, is highlighted as a means of influencing the seismic behaviour of new and existing building stock.

7. Acknowledgements

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