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Assessment of infill wall topology contribution in the overall response of frame structures under seismic excitation

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Abstract. This paper identifies the effects of infill wall existence and arrangement in the seismic response of steel frame structures. The methodology followed was based on the utilisation of overall seismic response indicators that distil the complexity of structural response in a single value hence enabling their straightforward comparative and statistical post process. The overall structure damage index after Park/Ang ($OSDI_{PA}$) and the maximum inter-story drift ratio (MISDR) have been selected as widely utilized structural seismic response parameters in contemporary state of art. In this respect a set of 225 Greek antiseismic code (EAK) spectrum compatible artificial accelerograms have been created and a series of non-linear dynamic analyses have been executed. Data were obtained through nonlinear dynamic analyses carried on an indicative steel frame structure with 5 different infill wall topologies. Results indicated the significant overall contribution of infill walls with a reduction that ranged 35-47% of the maximum and 74-81% of the average recorded $OSDI_{PA}$ values followed by an overall reduction of 64-67% and 58-61% for the respective maximum and average recorded MISDR values demonstrating the relative benefits of infill walls presence overall as well as localised with similar reductions observed in 1st level damage indicators.

Keywords: infill wall topology; dynamic analysis; overall structural damage; seismic response; nonlinear behaviour; frame structure

1. Introduction

Steel frame structures with infill walls are a rather common type of building type extensively utilized where speed of construction is of the essence. Infill walls, depending on the architectural considerations, cover the whole or part of one or more of the steel sub-frames. Infill walls are treated, during a structure's design stage either as part of the structural system or not. In practise the latter is usually selected due to the simplifications this method provides in the necessary calculations for the design of a structure. Furthermore, due to the very nature of steel structures of being able to adapt at many different type of use with different internal space arrangements, designers select to provide maximum transformability to the respective occupants by arranging the

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structural system in order to maximize free areas. As a result, in most cases steel structures are design and constructed based on the assumption of all horizontal actions to be accommodated by a frame resisting design.

In theory any post design stage infill wall shall be constructed in such a way as to avoid any load being transferred to them to avoid transmitting seismic load to such a brittle element as well as to avoid interference with the structure's design ductility. In practise this leads to the isolation of said infill walls from the structural system therefore negate any influence that might have arisen. This paper assessed the effect of infill wall arrangement into the overall seismic structural behaviour of a steel frame building based on the assumption that such isolation does not take place and steel frame structure is influenced.

Usually, when engineers study the structural characteristics of a frame structure chose to ignore the effect of infill walls taking only under consideration the extra weight they contribute in the load system without incorporating the structural rigidity that comes with such utilization. Paulay and Priestley (1992) proposed a theory about the seismic behaviour of masonry infilled frame calling for the alterations the existence of such infill elements bring to the structural system along with improved overall lateral load capacity. More contemporary works have utilized the bracing of steel moment resisting frames as a means of improving the existing seismic response characteristics with great results (Di Sarno 2009). On the other hand, from site surveys and both analytical and experimental analysis results, it is rather widely acknowledged that infill walls contribute in the modal response of the structure.

In accordance to the Federal Emergency Management Agency (FEMA) prepared FEMA 273 (ATC-33 1977) and NEHRP Guidelines for the Seismic Rehabilitation of Buildings provisions, after identifying the possible positive effects, dictates that concrete frames with infill walls must be constructed in such a way as to ensure infill element and frame interaction under design loads. The reasons behind such contribution is usually being related with the contribution of infill walls in the overall building structural rigidity, the structure's natural period and damping coefficient. In this paper the numerical relationship between infill wall existence in a steel frame and the overall structural response in case of seismic loading is investigated. Similarly, in steel moment resisting frames, a lot of effort has been spent in the research of the contribution of infill walls in their seismic characteristics.

The existence of infill walls in frame structures as well as their contribution to the seismic response has been a major point of study from various researchers in the past in an attempt to establish the relationship between the frame lateral loading capacity and the existence of frame. Work has been done on the issue of partially infilled frames where openings are present in an effort to address the issue of seismic behaviour on frame level (Albanezi *et al*. 2004; Mondal and Jain 2008) showing the significance of infill frame presence in frame structures. One notable exception to the above have been identified in the case of partial partial-height infill walls that often cause columns to experience non-ductile shear failures (short column effect) rather than respond in a ductile and predominately flexural manner as intended (Zarnic and Gostic 1997) while D'Ayala *et al*. (2009) highlighted the importance of proper modelling characteristics to ascertain the validity of the infill model damage propagation.

Regarding the type of infill wall material several different suggestions have been investigated in the past, each one with its own merits and limitations. Bruneau and Bhagwagar (2002) studied the effects of steel and other ductile materials as well as steel plates while Di Sarno and Elnashai (2009) focused in frame bracing, all showing improvement in the overall structural response. Of the aforementioned the most easily applicable and readily available material is masonry infill, therefore presenting a rather interesting solution for steel frame structure seismic rehabilitation despite the apparent shortcomings in terms of additional weight.

The extent of infill wall application as well as its individual frame coverage has been a point of extensive research in the past with restrained or partially restrained infill wall frames showing improved seismic characteristics (Sun *et al*. 2011) but with limited practical use for structural rehabilitation, due to the difficulty to achieve this kind of restrain, in frames that has not been designed for this purpose. Most recently Tasnimi *et al*. (2011), once more shown the beneficial contribution of both solid and non-solid infill walls in individual frames' seismic response while specific numerical models to study the above have been proposed (Mohebkhah *et al*. 2008). This kind of in depth analysis would be out of the scope of this paper that concentrates on the overall structural characteristics of steel moment resisting frame structures where the importance is shifted to a macro building-wise scale.

2. Methodology overview

Having established the beneficial effects of infill walls in the seismic response of frame structures the implementation of such a building arrangement becomes evident for both new structures and for retrofit purposes. As a caveat for the later one, being the necessity of proper research and mitigation of all implications such a post design intervention can have on the dynamic response characteristics of the structure. Therefore greater effort must be spent in the integration of the existing research into a generalized framework that will allow researchers to assess and quantify not only the effects of the existence of infill walls in a frame structure but also account for the different possible topologies that can be realised. This highlights the importance of the possibility to identify the effect of the infill wall topology in the context of the overall structural seismic response as a mean to better address the issues of design and retrofit according to structural and architectural needs or constrains. The solution to such a research question can be better facilitated with the development and application of a methodology that remains essentially agnostic of the micro scale seismic effects, on an element basis, but rather focussed on the overall damage distribution and seismic behaviour.

This research was based into addressing the issue of the overall seismic behaviour in a way that will account for the differences in alternative infill wall topologies and provide the response characteristics of a frame structure in an effort to empower researchers to achieve the optimum infill wall distribution based on the design requirements and constraints. Since the main objective for this paper was to provide a framework rather than a specific study of the infill wall contribution in seismic structural response and because the methodology presented is intended to act as a research tool for the identification of the intervention guidelines for potential seismic rehabilitation. All conditions selected were based on merit of generalization and in accordance with the researchers' intention to proceed towards the creation of an assessment methodology that could, according to research and implementation needs, with the necessary modifications, address the overall structural seismic response for a variety of structural types; infill elements; and seismic conditions.

This paper identifies and quantifies the effects of infill wall existence and arrangement in the seismic response of steel frame structures highlighting their potential seismic design significance. To achieve the above, several artificial accelerograms compatible with the Greek Antiseismic Code (OASP 2003) have been composed and a nonlinear dynamic analysis has been carried out to provide the structural response for the given seismic excitations. The overall structure damage index after Park/Ang ($OSDI_{PA}$) and the maximum inter-story drift ratio (MISDR) have been selected as some of the most widely utilized structural seismic response parameters in contemporary state of art as well as their storey level equivalents (LDI_{PA} and LISDR) to assess the more regionalized seismic behaviour as well.

For the structure under investigation, the creation of a simple analytical model of a typical commercial steel frame 10 storey building and the application of 4 different infill wall layouts resulting into 5 different structure types was realised. All structural elements and connections were design 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 (CEN 1993) and EC8 (CEN 2004) respectively, hence representing a typical contemporary steel structure.

The use of spectrum compatible artificial accelerograms was selected to enable the production of a wide range of response data that share a common ancestry and allowed for executing a range of comparative studies between them, something not possible if naturally occurring ground motions were utilized. In light of the above, a set of 225 Greek antiseismic code (EAK) spectrum compatible artificial accelerograms have been created to assess the behaviour of the aforementioned models in a wide range of seismic excitations in their operating environment and a series of non-linear dynamic analyses have been executed to record this behaviour.

3. Synthetic accelerograms

The seismic excitations used for the dynamic analyses in this study are based on artificial accelerograms created to be compatible with the design spectra of the current Greek antiseismic code. The reason for choosing this approach rather than relying on natural accelerograms was dictated by the need for a sufficiently large statistically robust database. In order to bypass these limitations regarding the statistical coherence and wide spread of recorded structural damage the creation of several artificial accelerograms were created. Compatible strong motion acceleration time-histories have been created with the use of suitable techniques that produced compatible accelerograms that matched 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 upon was used. The creation of a large specimen of the aforementioned accelerograms was realized, in order to provide a sufficiently big and statistically coherent set of seismic data. For the creation of such artificial accelerograms, the program SIMQKE-GR (Gelfi 2006) relying on the property of every periodic function to be analysed into a finite set of sinusoidal waves, has been utilised.

With the use of a differentiated choice of seismic parameters 225 artificial accelerograms have been created, all compatible with the Greek antiseismic code (OASP 2003) 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 0.16 g, 0.24 g and 0.36 g). All the above were based on the assumption of category B subsoil (deep deposits of medium dense sand or over-consolidated clay at least 70m thick), as described in EC 8 and the Greek antiseismic Code.

4. Damage indices

As explained previously, attention is focused on damage indicators that consolidate all member damage into one single value that can be easily and accurately be used for the statistical exploration of the interrelation with the also single-value seismic parameters in question. Thus, in the OSDI model after Park/Ang (Park and Ang 1985) the global damage is obtained as a weighted average of the local damage at the ends of each element. The local damage index is given in Eq. (1.1).

$$DI_L = \frac{\theta_m - \theta_r}{\theta_u - \theta_r} + \frac{\beta}{M_y \theta_u} E_T \tag{1.1}$$

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 a strength degrading parameter; My 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 after Park/Ang is presented in Eq. (1.2).

$$OSDI_{PA} = \frac{\sum_{i=1}^{n} DI_L E_i}{\sum_{i=1}^{n} E_i} \tag{1.2}$$

where, $OSDI_{PA}$ is the global damage index after Park/Ang; DI_L the local damage index after Park/Ang,; E_i the energy dissipated at 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 in each respective level, providing a local damage index relevant to each separate level as shown in Eq. (1.3).

$$LDI_{PA} = \frac{\sum_{i=1}^{n} DI_{LL} E_{iL}}{\sum_{i=1}^{n} E_{iL}} \tag{1.3}$$

where, LDI_{PA} is the level structural damage index after Park/Ang; DI_{LL} the local damage index after 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.

The maximum inter-storey drift ratio (MISDR), is believed to accurately depict the recorded post seismic level of structural and architectural damage of a structure alike. The correlation of MISDR with the above has repeatedly been proven both experimentally as well as from post-earthquake site surveys in areas where catastrophic seismic events took place (Gunturi and Shah 1992) and is widely recognized as an effective tool of damage representation. Furthermore, MISDR is simple in its calculation, as the maximum observed value throughout the recorded individual inter-storey drift ratio of each level (LISDR) given in Eq. (2).

$$LISDR_i = \frac{u_i - u_{i-1}}{h_i} 100 \ [\%] \tag{2}$$

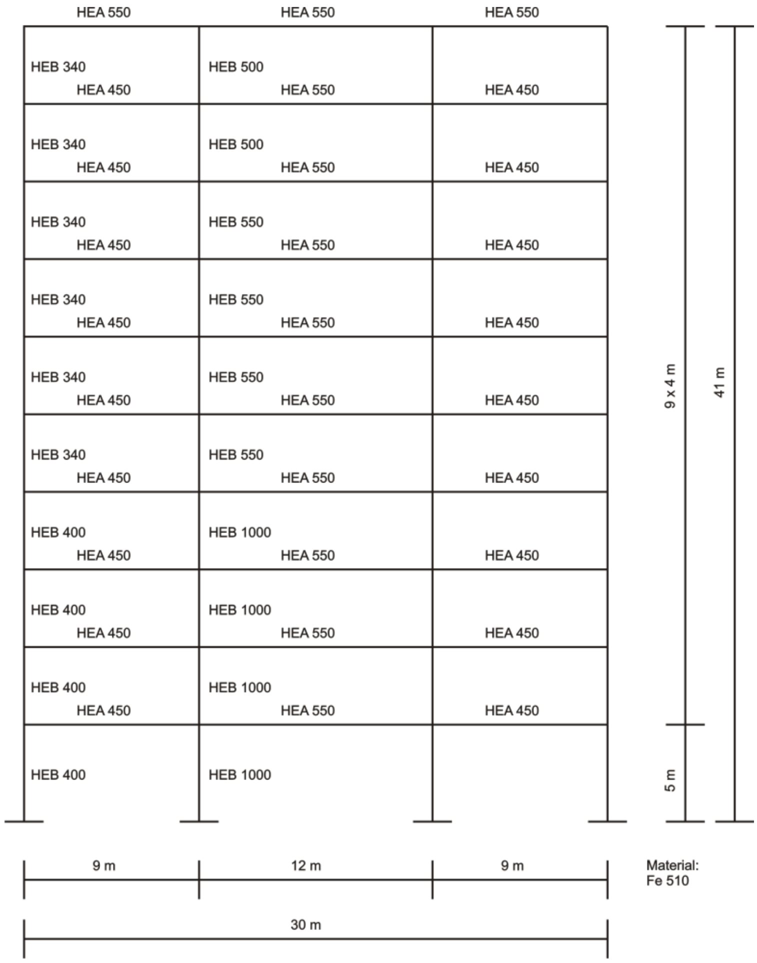


Fig. 1 Bare steel frame structure (Frame 0)

where, $LISDR_i$ is the level inter-storey drift ratio; u_i is the drift of floor i; u_{i-1} is the drift of floor i-1, h_i is the height of floor i and MISDR is the maximum recorded value amongst the total amount of storeys.

5. Numerical application

The geometry, layout and the structural elements profiles of the chosen 10 storey building for Frame 0 (Bare frame), Frame 1 (2 outer bays bearing infill walls), Frame 2 (central bay bearing infill wall), Frame 3 (same as Frame 1 but with no infill wall present at ground level) and Frame 4 (same as Frame 2 but with no infill wall present at ground level) are given in more detail in Fig. 1 and 2. The selection of Frame 3 and Frame 4 was to cover a large portion of commercial buildings that accommodate parking or other commercial activities in the ground elevation that architecturally prohibit infill wall installation. Structural detailing was completed by implementing the requirements of both Eurocode 3 (CEN 1993) and the current Greek antiseismic code (OASP

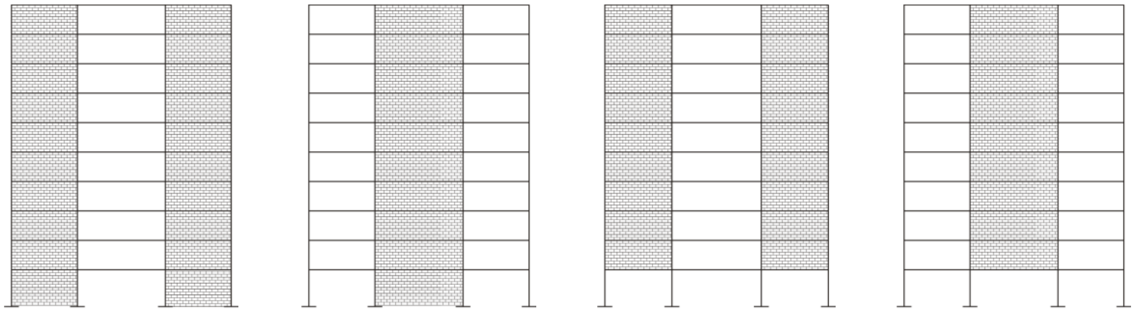


Fig. 2 Alternative infill wall arrangements (Frame 1 to 4)

2003) for steel anti-seismic structures. The slabs' thickness has been designed to be 20cm. The whole design was 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=0.16 g) according to the Greek antiseismic code. In addition, live, snow and wind loads have also been taken into account as well as the eccentricity of structural element from verticality. The numerical values of loads, safety factors as well as load combinations have been chosen in accordance with Eurocode 1, 3 and the Greek antiseismic code requirements.

The next step is the creation of the alternative patterns of infill walls to be studied, as presented in Fig. 2. Namely, Frame 1 that constitute infill wall present in the two 9m wide corner frames at the full height of the building and Frame 2 that comprising of a single column of infill walls in the middle 12m frames; as well as their no infill wall ground elevation frame counterparts named Frame 3 and Frame 4 respectively. The Infill wall topology selection has been made under the guidelines of usual construction practise. In that light symmetrical infill wall topologies have been selected along with the existence or not of a "pilotis" design for the ground-floor, a usual construction practise in the highly seismic areas of Greece and Turkey employed to enhance the structure's everyday operation and maximise its usability. Those different topologies have been selected to enable researchers into identifying possible structural behaviour relevance in terms of both total infill wall element area as well as non-uniformity of infill wall elements in elevation with the inclusion of Frame 3 and 4 "pilotis" based examples that might exacerbate any kind of soft storey effect.

With the design procedure of the frame structures completed and by the implementation of nonlinear dynamic analysis utilizing the accelerograms created, an evaluation of the structural seismic response of all frames using the computer program IDARC2D (Reinhorn *et al*. 2009) has been realised. In this regard, a three-parameter Park model was used to specify 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, specifies the parameter values of the above degrading parameters. This study used the nominal parameter for stiffness degradation throughout.

IDARC2D utilises the Newmark-β method of numerical integration followed by Newton/Raphson's method for root approximation 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 Walls has been incorporated in the model in the form of diagonal compression struts in the respective sub-frames. The analytical model used in the present study assumes that the contribution of the infill panel to the response of the infilled steel frame can be modelled by replacing the panel with a system of two diagonal masonry compression struts (Madan *et al*. 1997, Reinhorn *et al*. 2009). It takes into account the nonlinear behaviour of infilled frames, considering the limited ductility of infill material. Thus, the stress-strain relationship for masonry in compression is idealized as an increasing polynomial function until the peak stress is reached for a given strain. For higher strains, the stress drops with increasing strains to a small fraction of the peak value where after the stress remains almost constant at this value. Since the tensile strength of masonry is negligible, the individual masonry struts are considered to be ineffective in tension. However, the combination of both diagonal struts provides a lateral load resisting mechanism for positive as well as negative directions of loading.

As IDARC2D adopts an equivalent two diagonal compression strut model for the analysis of steel frames with taking into account the elastoplastic behaviour of infilled frames and the limited ductility of infill materials. The investigation of localised effects cannot be studied using the applied methodology. This research focussed in overall structural behaviour characteristics (i.e., change in natural frequency, Maximum interstorey drift ratio, global damage index after Park and Ang, etc.). In that regard, local effects have been incorporated in the global stiffness model but cannot be directly analysed. In effect, although one can account stiffness reduction, as recorded in stiffness degradation, in crack formation and propagation the employed technique is not in position to identify their amount, length or width. The Macro Modelling approach used in the present study considers the entire panel as a unique element, a technique that allows for adequate evaluation of the non-linear force-deformation response of the structure and individual components under seismic loading where the computed force-deformation response may be used to assess the overall structure damage and its distribution to a sufficient degree of accuracy (Reinhorn 2009).

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 (Reinhorn *et al*. 2009). For all recorded non-linear analyses, the maximum inter-storey drift ratio (MISDR) and the overall structural damage index after Park/Ang ($OSDI_{PA}$) (Park and Ang 1985) have been evaluated as widely accepted direct methods of post seismic structural damage evaluation, based on the simplicity and straightforwardness of their calculation. The above selection was made in order to cover both the structural damage due to deformation (MISDR) but also the effects of the combination of deformation and hysteretic energy absorption ($OSDI_{PA}$). Fig. 3 provides an analysis procedure flowchart that covers the above and demonstrates the logic and computational process.

6. Results and discussion

The beneficial influence of the infill wall presence in the frame structure can be observed in great detail through detail investigation of the effects of a randomly selected ground motion as demonstrated in Tables 1 and 2 as well as in Fig. 4. The numerical results evaluated by nonlinear dynamic analyses for all the examined frames with the use of a Greek antiseismic code (OASP, 2003) response spectrum compatible derived artificial accelerogram. The used seismic excitation corresponds to a PGA of 0.3g, with a total duration (TD) of 30 s, selected to be compatible with a

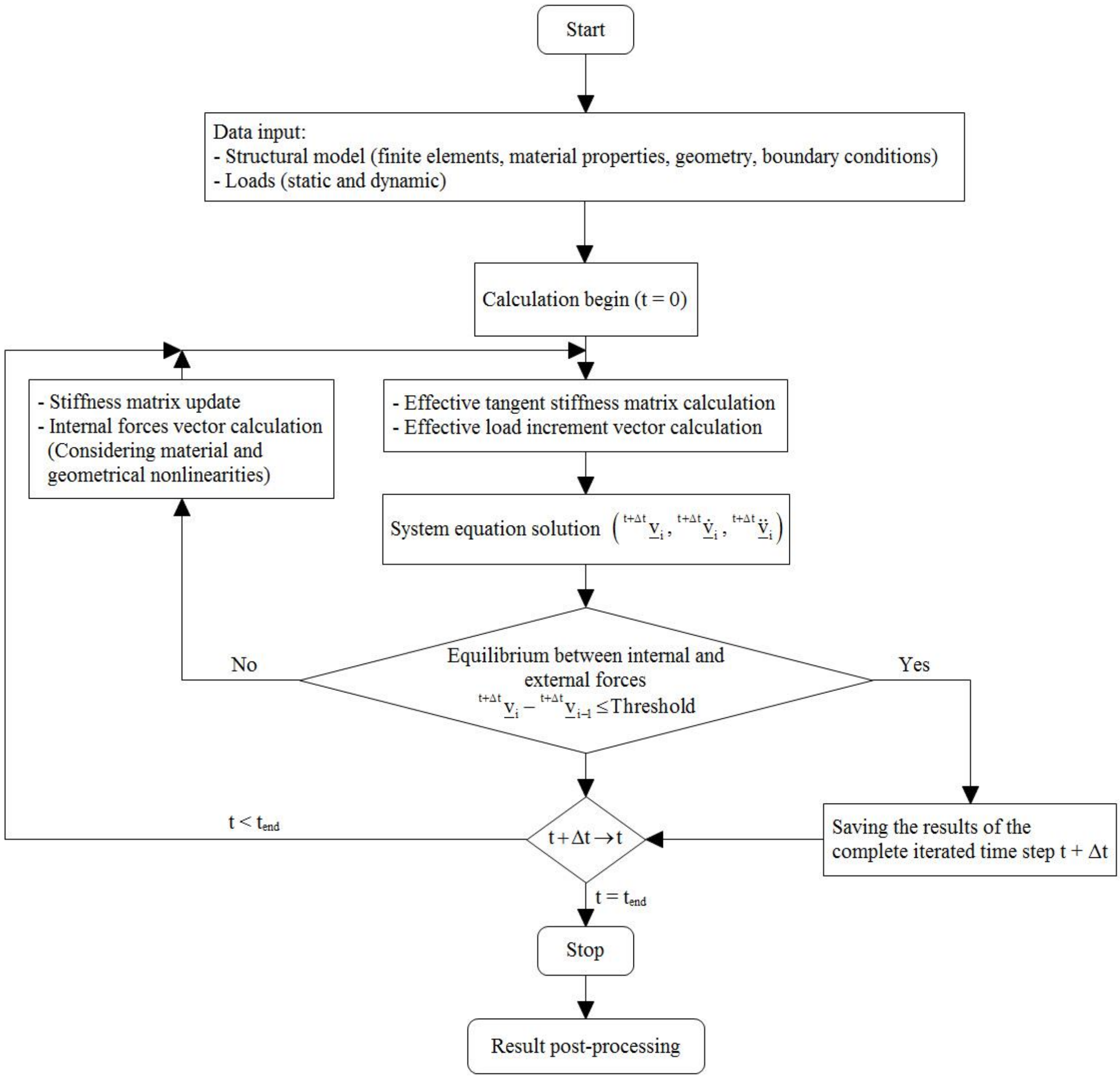


Fig. 3 Analysis procedure flowchart for the software IDARC2D employed

nominal design spectra acceleration (α) equal to 0.24g and is based on the assumption of category B subsoil, as described in EC 8 (CEN 1993) and the Greek antiseismic Code (OASP 2003).

For the above, Table 1 shows the maximum positive and the minimum negative moments as recorded at the base of each ground floor column, along with the respective percentage reduction observed with the introduction of infill wall elements. On the other hand Table 2 shows the maximum recorded values of shear forces response for the column bases at ground level, along with the respective percentage reductions observed between the bare frame and the different infill topologies examined.

In the same spirit, Fig. 4 shows the shear force time-history at the ground floor for Frames 0 and Frame 1 dynamically excited by the previously described artificial accelerogram.

Table 1 Maximum moments and % reductions recorded at ground floor column base

		Frame	Left	Middle Left	Middle Right	Right	Sum	(%) Reduction against Frame 0				
Recorded moment at ground floor column base [kNm]	Maximum	0	403.20	2834.00	2844.0	505.20	6586.4	Left	Mid L	Mid R	Right	Sum
		1	146.40	1777.00	1884.0	246.30	4053.7	63.7	37.3	33.8	51.3	38.5
		2	162.70	1932.00	2039.0	262.60	4396.3	59.7	31.8	28.3	48.0	33.3
		3	168.90	1986.00	2092.0	268.80	4515.7	58.1	29.9	26.4	46.8	31.4
		4	177.20	2067.00	2173.0	277.20	4694.4	56.1	27.1	23.6	45.1	28.7
	Minimum	0	-452.60	-2806.0	-2796.0	-350.00	-6404.6	Left	Mid L	Mid R	Right	Sum
		1	-215.50	-1597.0	-1490.0	-115.60	-3418.1	52.4	43.1	46.7	67.0	46.6
		2	-279.30	-2187.0	-2080.0	-179.40	-4725.7	38.3	22.1	25.6	48.7	26.2
		3	-236.60	-1789.0	-1682.0	-136.70	-3844.3	47.7	36.2	39.8	60.9	40.0
		4	-294.80	-2330.0	-2223.0	-194.90	-5042.7	34.9	17.0	20.5	44.3	21.3

Table 2 Percentage change of minimum negative moments at the bottom of the ground floor columns

	Frame	Ground Floor Column	(%) Reduction against Frame 0
Maximum Base Shear Force [kNm]	0	1418.82	Ground Floor Column Mid L Mid R Right Sum
	1	950.51	33.0
	2	1086.81	23.4
	3	970.03	31.6
	4	1098.73	22.6

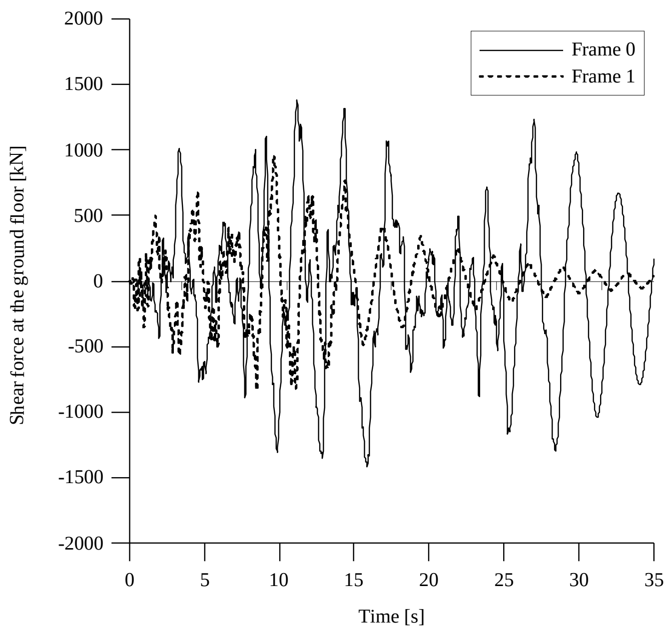


Fig. 4 Shear force time-history at the ground floor for Frames 0 and 1

All these results substantiate the advantageous influence of the infill wall presence. Hence, the frames with infill walls have in all cases lower absolute moment and shear force values in the ground floor (Tables 1 and 2). This is further validated from the differences presented in the recorded shear force time-histories between the bare and infilled frames (in that case Frame 1). Furthermore, there is a significant percentage value reduction of both shear force and recorded moment at the base of the ground floor columns ranging from 17% to 67%. Finally, the results show that infill wall presence in the ground floor (Frame 1 and Frame 2) has a positive effect on the response quantities in comparison with the case where infill wall is absent in the ground floor (Frame 3 and Frame 4).

To positively identify the effect of infill wall topology in the seismic response of the steel frame a comparative study has been carried out focusing on the recorded differences between the 5 frames studied as those are described with the selected damage indices. In this regard several different set of results have been considered in order to appreciate the obfuscation effect of the excessive zero values that has been recorded in the $OSDI_{PA}$ and 1st Level DI_{PA} (1st LDI_{PA}) for all infill bearing frame typologies. It has therefore been selected to record the differences in three distinct result groups based in the recorded $OSDI_{PA}$ values. The three groups are the total (225 cases) number of results; the results where at least one of the infilled frames recorded non-zero $OSDI_{PA}$ (67 cases) and finally an individual account (variable number of cases) of non-zero $OSDI_{PA}$ values for each frame typology.

The aforementioned results are summarised in Table 3 for the differences between frame typologies in respect to the recorded overall damage indices for the total amount of results; Table 4 and Fig. 5 presents the recorded differences in respect to the 1st level damage indices for the whole result spectrum; Table 5 and Fig. 6 refer to the recorded differences for all cases that register at least one non-zero $OSDI_{PA}$ value across the different frame typologies and finally Table 6 and Fig. 7 where all non-zero $OSDI_{PA}$ values are considered for each individual frame typology.

In more details, Table 3 presents the maximum, minimum and average values of the overall structural damage indices as well as their rate of change in percentage in respect to the bare frame (Frame 0). Therefore, we can see that the infill wall sporting frames present a reduction of up to approximately 46.8% in the maximum $OSDI_{PA}$ values recorded when compared to the bare frame with the reduction in the median values reaching values in the region of 81.1%. Similar results have been recorded for the maximum MISDR values where a reduction of approximately 67.4% can be observed while for the average values this is in the region of 60.7%.

Table 3 Statistical values and average reduction of $OSDI_{PA}$ and MISDR results recorded

	Frame	Min	Max	Average	(%) Reduction against Frame 0			
$OSDI_{PA}$	0	0.017	0.304	0.188	Frame	Min	Max	Average
	1	0.000	0.162	0.035	1	100	46.8	81.1
	2	0.000	0.175	0.037	2	100	42.3	80.4
	3	0.000	0.197	0.049	3	100	35.1	74.1
	4	0.000	0.191	0.047	4	100	37.3	74.8
MISDR %	0	0.612	2.718	1.408	Frame	Min	Max	Average
	1	0.220	0.892	0.552	1	64.1	67.2	60.8
	2	0.242	0.958	0.585	2	60.5	64.8	58.5
	3	0.224	0.884	0.553	3	63.4	67.5	60.7
	4	0.248	0.952	0.587	4	59.5	65.0	58.3

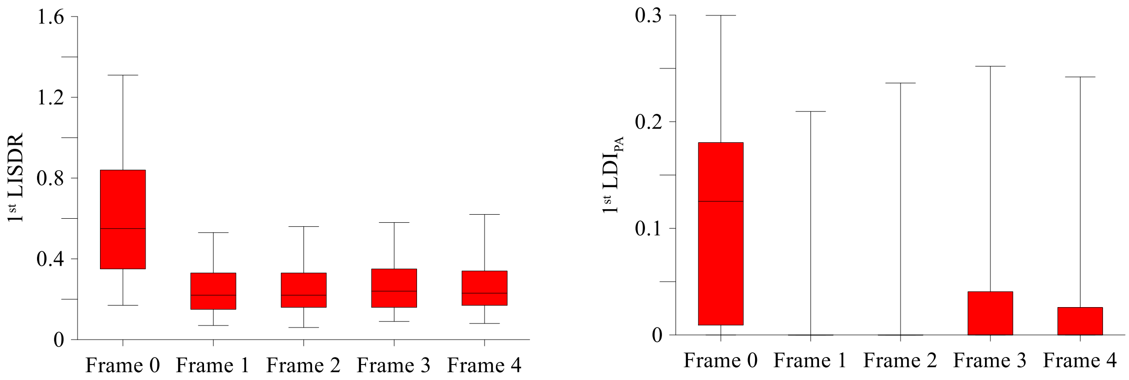
Table 4 Infill frame 1st Level ISDR and DI_{PA} results statistical values for all results (225 cases)

	1st LISDR	Frame 0	Frame 1	Frame 2	Frame 3	Frame 4
1st LISDR	Average	0.613	0.240	0.250	0.262	0.266
	Median	0.55	0.22	0.225	0.24	0.235
	Variance	0.078	0.011	0.012	0.013	0.014
	Std. dev.	0.280	0.103	0.110	0.114	0.118
	CoV	45.6%	42.8%	43.7%	43.4%	44.5%
1st LDIPA	Average	0.112	0.020	0.023	0.037	0.036
	Median	0.126	0.0	0.0	0.0	0.0
	Variance	0.007	0.002	0.004	0.005	0.005
	Std. dev.	0.086	0.046	0.050	0.069	0.069
	CoV	76.2%	225.3%	222.8%	185.5%	192.8%

In all cases the biggest reduction can be observed in the case where infill walls are more present (Frame 1) indicating the positive contribution of these non-structural elements in the overall building behaviour. As the numerical results have shown infill walls proved to have positive contribution in the structure's seismic response giving us reduced values of the mean DI_{PA} and further more decrease can be recorded in the non-zero mean values of the same DI between the bare frame structure and its infill wall reinforced twins. Similar results have been observed for the MISDR as well, where reduction has been noted in all infill walls "reinforced" cases as opposed to the bare frame. Finally, regarding the differentiation between the MISDR evaluated in the alternative topology cases there is an emerging pattern showing that the amount of infill walls is more important than their continuity, when the results between the infill wall bearing frames are examined. The difference between them is in the region of 3% which further enhances the conviction that existence of infill walls is plays a major role in the overall response of a frame building structure for artificial accelerograms compatible with a response spectrum based on a nominal ground acceleration a=0.16 g, 0.24 g and 0.36 g.

In order to fully investigate the effect of infill walls in the overall seismic structural behaviour the analysis of the differences between different arrangements of infill walls in terms with the first-storey localised behaviour damage as expressed by the 1st level damage index after Park/Ang (1st LDI_{PA}) and the 1st level interstorey drift ratio (1st LISDR). The above become particularly important due to the increased weight, averaging 65% towards the total damage value observed from the recorded results. This has been achieved with a separate study focussing on the investigation of the localized structural damage indices and their behaviour towards the overall results. The work has been split into 3 distinct steps covering a variety of behavioural value data as described in the next few paragraphs.

The first part of the investigation addresses the 1st LDI_{PA} and 1st LISDR behaviour against all studied frame arrangements taking into account the full spectrum of results. The statistical results for this are summarised in Table 4 for 1st LISDR and 1st LDI_{PA}, Fig. 5 present the relevant box and whiskers chart to illustrate the recorded values. As indicated by the statistical summary, 1st LISDR values do present some striking analogies in terms of coefficient of variation (CoV) between the different frame arrangements as well as some important differentiations in terms of data cohesion demonstrated by the proximity of the bulk of results in terms of the average value. Those results present not only an improved, in terms of 1st LISDR reduction, behaviour but also lower results scatter, suggesting an overall improvement in structural response definition. During the statistical



(a) (b)

Fig. 5 Bar chart representation of 1^{st} Level ISDR (a) and DI_{PA} (b) from Table 4

Table 5 Recorded 1^{st} level DI_{PA} and ISDR reduction between frames (all data, 225 cases)

			Frame 0 to 1	Frame 0 to 2	Frame 0 to 3	Frame 0 to 4	Frame 1 to 3	Frame 2 to 4	Frame 1 to 2	Frame 3 to 4
1^{st} Level	ISDR	Average	60.8%	59.2%	57.2%	56.6%	8.4%	6.0%	4.0%	1.4%
		Minimum	58.8%	64.7%	47.1%	52.9%	22.2%	25.0%	-16.7%	-12.5%
		Maximum	59.5%	57.3%	55.7%	52.7%	8.6%	9.7%	5.4%	6.5%
	DI_{PA}	Average	81.9%	79.9%	66.8%	68.2%	45.4%	36.8%	9.8%	-4.5%
		Minimum	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A
		Maximum	30.1%	21.2%	15.9%	19.3%	16.8%	2.4%	11.3%	-4.1%

analysis of $1^{st}OSDI_{PA}$ results and due to the increased amount of zero values involved, although a significant reduction in recorded values has been observed the statistical aspect of those results needs to be further investigated. In terms of the seemingly high coefficient of variation (CoV) values recorded for 1^{st} LDI_{PA}, it can be squarely attributed to the multitude of zero and extremely low values that have been recorded throughout and demonstrated in Fig. 3(b) due to the vast overall structural behaviour improvement in agreement with the 1^{st} LISDR results.

Fig. 5 presents in the form of box and whiskers an overview of the statistical characteristics of 1^{st}LISDR and $1^{st}LDI_{PA}$ respectively. The aforementioned reduction in the recorded values is easily demonstrated along with the statistical characteristics regarding the result dispersion presenting the minimum and maximum values (whiskers) as well the lower and upper quartiles (box) and the median (straight line marking in the box) values.

From Fig. 5 the improvement of 1^{st} level behaviour, in terms of the selected damage indicators, is evident not only by the actual reduction observed but also with the robust concentration of the values and the reduced scatter, in respect to the bare frame recorded ones. The overall positive contribution of the selected infill wall topologies has been demonstrated by the apparent reduction observed in both 1^{st}LISDR and 1^{st} LDI_{PA} values. The recorded reduction for each individual frame has been evaluated and summarized in Table 5 showing a notable mean 1^{st}LISDR reduction of 58.2% in the values recorded between the bare and infill bearing frames as well as a 22% to 25% of reduction of the minimum recorded values between the total height infill wall bearing frames and their no infill wall ground elevation frame counterparts. The results are even more prominent in the case of 1stLDI$_{PA}$ value investigation where a vast improvement in terms or damage indices values is recorded with an average reduction of 75% between the bare and infill wall bearing frames. Similarly, a 45.4% and 36.8% of reduction has been recorded in the values of the total height infill wall bearing frames and their no infill wall ground elevation frame counterparts.

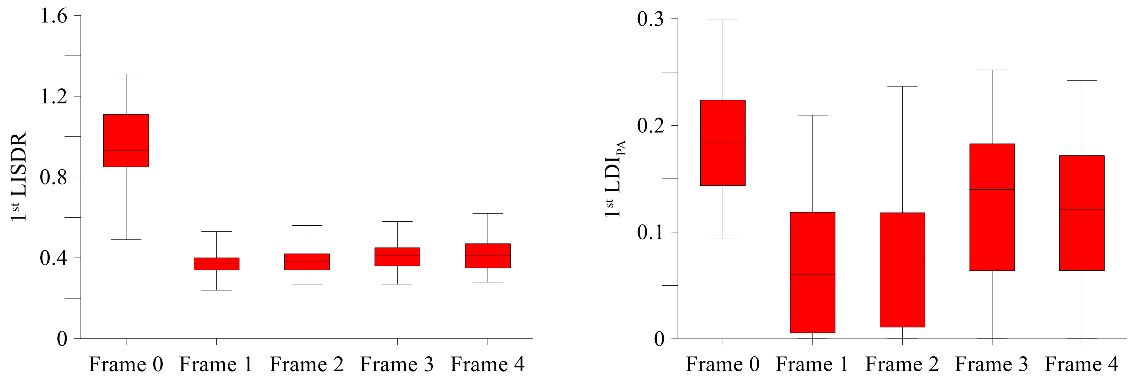
Data in Table 5 suggests the close relationship between the 1st level and overall results supported by the comparatively similar behaviour of the recorded value reduction observed in OSDI$_{PA}$ and MISDR indices presented in Table 3 for each individual infill wall arrangement against the bare frame.

Nevertheless, due to the significant amount of 1st LDIPA values reduced to zero and the induced extreme result skew presented in Tables 4 and 5 and demonstrated in Fig. 5b the selection of cases where at least one non-zero 1st LDIPA value between Frame 1 to 4 has been recorded was investigated. A total amount of 67 cases has been identified, utilized and their results presented in Table 6.

As expected through the elimination of most cases presenting zero 1st LDI$_{PA}$ value, a significantly reduced CoV has been recorded indicating the more robust and less scattered nature of the results. An indication that can be further established by a study of Fig. 6(b)

Table 6 Infill frame 1st Level ISDR and DI$_{PA}$ results statistical data with one non-zero OSDI$_{PA}$ record across frames (67 cases)

	1st LISDR	Frame 0	Frame 1	Frame 2	Frame 3	Frame 4
1st LISDR	Average	0.961	0.374	0.390	0.411	0.417
	Median	0.930	0.370	0.380	0.410	0.410
	Variance	0.027	0.003	0.005	0.004	0.006
	Std. dev.	0.165	0.053	0.068	0.066	0.080
	CoV	17.1%	14.2%	17.5%	16.0%	19.2%
1st LDI$_{PA}$	Average	0.189	0.069	0.076	0.125	0.119
	Median	0.184	0.060	0.073	0.140	0.121
	Variance	0.003	0.004	0.005	0.005	0.006
	Std. dev.	0.052	0.062	0.067	0.071	0.077
	CoV	27.4%	90.7%	88.9%	57.3%	64.2%



(a) (b)

Fig. 6 Bar chart representation of 1st Level ISDR (a) and DI$_{PA}$ (b) from Table 6

Table 7 Recorded 1st level DI_{PA} and ISDR reduction between frames (one non-zero $OSDI_{PA}$ record across frame types, 67 cases)

			Frame 0 to 1	Frame 0 to 2	Frame 0 to 3	Frame 0 to 4	Frame 1 to 3	Frame 2 to 4	Frame 1 to 2	Frame 3 to 4
1st Level	ISDR	Average	61.1%	59.4%	57.3%	56.6%	8.9%	6.5%	4.1%	1.5%
		Minimum	51.0%	44.9%	44.9%	42.9%	11.1%	3.6%	11.1%	3.6%
		Maximum	59.5%	57.3%	55.7%	52.7%	8.6%	9.7%	5.4%	6.5%
	DI_{PA}	Average	63.9%	60.0%	33.9%	36.7%	45.4%	36.8%	9.8%	-4.5%
		Minimum	100.0%	100.0%	100.0%	100.0%	N/A	N/A	N/A	N/A
		Maximum	30.1%	21.2%	15.9%	19.3%	16.8%	2.4%	11.3%	-4.1%

Table 8 Infill frame 1st Level DI_{PA} results statistical data for non-zero $OSDI_{PA}$ results (variable cases)

$1^{st}LDI_{PA}$	Frame 0	Frame 1	Frame 2	Frame 3	Frame 4
Count	173	52	52	62	58
Average	0.145	0.088	0.097	0.135	0.138
Median	0.148	0.085	0.092	0.143	0.130
Variance	0.005	0.003	0.004	0.004	0.004
Std. dev.	0.068	0.056	0.061	0.064	0.065
CoV	46.9%	64.3%	62.2%	47.8%	47.0%

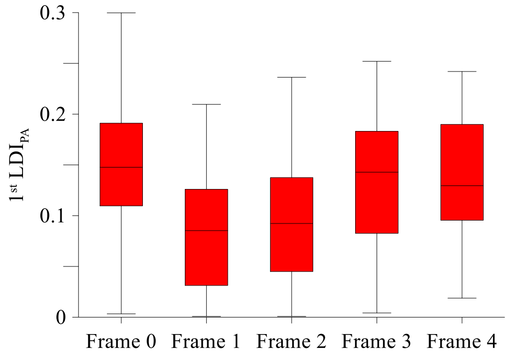


Fig. 7 Bar chart representation of 1st Level DI_{PA} from Table 8

The results clearly indicate a significant reduction of the 1st level index values underlining the significance of infill wall presence in the overall seismic behaviour of the structure. In Table 7 the improvement recorded is summarised as the percentage of value reduction observed between the different types of infill wall arrangements is presented. The reduction in terms of 1st LISDR values average 58.6% for the infill wall bearing structures in respect to the bare frame counterpart while a 7.7% reduction in the average 1st LISDR values has been recorded for the full height infill wall bearing frames Frame 1 and Frame 2 against Frame 3 and Frame 4 respectively that considered an open ground level frame arrangement. In terms of 1st LDI_{PA} reduction, similar reduction characteristics can be observed with an average reduction of 62% for the full height infill wall structures and a 35.3% average reduction for the bare ground elevation frame arrangements. The difference suggest the importance, in structural terms, of the uniform presence of infill walls as well as the expected overall improvement observed from the frame – infill wall interaction.

Table 9 Recorded 1st level DI_{PA} and ISDR reduction between frames (non-zero $OSDI_{PA}$ record, variable cases)

1st LDI_{PA}	Frame 0 to 1	Frame 0 to 2	Frame 0 to 3	Frame 0 to 4	Frame 1 to 3	Frame 2 to 4	Frame 1 to 2	Frame 3 to 4
Average	39.6%	33.1%	7.3%	5.1%	34.9%	29.5%	9.8%	2.3%
Minimum	75.5%	75.5%	-27.1%	-46.3%	80.8%	95.7%	0.0%	77.7%
Maximum	30.0%	21.2%	15.9%	19.3%	16.8%	2.4%	11.3%	-4.1%

The above observations regarding the behaviour highlight the importance of investigating the actual structural damage behavioural characteristics in a non-obfuscated manner. To achieve the above, further reduce the effect of zero 1st LDI_{PA} values and establish a clear trend towards the selected building' behavioural pattern in structural terms a selection of all cases with non-zero 1st LDIPA values was performed and the relevant statistical results of this data group are presented in Table 8 and Fig. 7.

The similarities of the 5 frames data groups, in terms of statistical characteristics, presented in Table 8 and Fig. 7 indicate the recordable improvement of infill wall steel frame structures over the bare frame one but also demonstrate the difference between similar infill wall arrangements. Those differences can be quantified by observing an 39.6% reduction of the average value decrease recorded for Frame 1 over Frame 0 as well as the 33.1% reduction recorded for the values of Frame 2 over Frame 0; while results for Frame 3 and Frame 4 reductions where 7% and 5.1% respectively; clearly demonstrating the quantifiably greater improvement associated with full height infill wall arrangements in terms of 1st LDI_{PA} as and presented in Table 9.

The results, presented a significant improvement in seismic structural behaviour is recorded when infill walls are utilized in comparison to the bare frame structure with an overall reduction of approx. 60% in MISDR values for all infill wall arrangements with comparable reduction in 1st level ISDR values ranging from 61% to 56.6% according to the structures' respective infill wall topology. Furthermore, measurable differences have been observed between different infill wall arrangements and damage distribution characteristics. In detail, an 80% and 74.5% reduction of $OSDI_{PA}$ values for the full height infill wall and the bare 1st elevation frame structural models respectively were recorded with comparable reduction of the 1st level ISDR values between 81% and 67.5%, for the full height infill wall and the bare 1st elevation frame structural models, while a respective reduction of 62% and 35.3% has been recorded when only non-zero $OSDI_{PA}$ cases where considered.

The overall beneficial influence of infill wall existence in steel frame structures has been investigated for both seismic structural behaviour and damage distribution characteristics. The importance of infill wall arrangement has also been explored and the benefits of each arrangement have been identified. It has been therefore concluded that the existence of infill walls benefits the structure in its overall seismic structural behaviour in a similar manner irrespectively of the particular differences in infill wall arrangement. On the other hand, structural damage distribution characteristics, as expressed by $OSDI_{PA}$, seem to be more sensitive to infill wall arrangement and linearity rather than shear area of infill walls favouring the full height infill wall arrangements due to the omission of structurally weak regions such as the bare frame on arrangement of the 1st level.

7. Conclusions

This paper quantified the influence of the topology of infill walls of steel frames with a bare ground floor on the seismic structural damage for each of those separate cases. A set of 225 artificial accelerograms have 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) after Park/Ang (DI_{PA}) and the maximum inter-story drift ratio (MISDR) while statistical analyses showed the strong interdependencies in respect to the alternative infill walls' topologies.

All presented numerical results showed a significant reduction in the overall damage indices of all infill wall topologies selected to be studied against the bare frame structure demonstrating the positive effect of infill walls in a steel frame structure as the one in question. 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 of approx. 60% in MISDR values for all infill wall arrangements with comparable reduction in 1st level ISDR values ranging from 61% to 56.6% according to the structures' respective infill wall topology. Furthermore, measurable differences have been observed between different infill wall arrangements and damage distribution characteristics. In detail, an 80% and 74.5% reduction of $OSDI_{PA}$ values for the full height infill wall and the bare 1st elevation frame structural models respectively were recorded with comparable reduction of the 1st level ISDR values between 81% and 67.5%, for the full height infill wall and the bare 1st elevation frame structural models, while a respective reduction of 62% and 35.3% has been recorded when only non-zero $OSDI_{PA}$ cases where considered. Based on the above the possibility for development of the necessary engineering framework, addressing structural design optimization with the use of non-structural elements, is highlighted as a means of improving the existing steel frame structure stock's seismic behaviour. The above, warrants the execution of diverse infill wall pattern arrangements to investigate the point of diminishing reward in terms of MISDR or $OSDI_{PA}$ reduction according to the structure's individual occupancy / architectural requirements.

It is therefore the conclusion of this work that infill walls can play an important role in steel frame buildings seismic behaviour both with their inclusion in the original design or when utilized as a seismic retrofit to improve a structures characteristics. This positive result has been recorded in all examined cases under investigation and quantifiable improvement has been demonstrating without imposing significant risks of localized damage concentration in, the most affected by those changes, 1st level elevation. Based on the above the possibility for development of the necessary engineering framework, addressing structural design optimization with the use of non-structural elements, is highlighted as a means of improving the existing steel frame structure stock's seismic behaviour.

References

Albanesi, S., Albanesi, T. and Carboni, F. (2004), "The influence of infill walls in RC frame seismic response", *High Performance Structures and Materials II*, Eds. Brebbia, C.A. and De Wilde W.P., WIT Press, 621-630.

ATC 33 (1997), "NEHRP Guidelines for the Seismic Rehabilitation of Buildings", Federal Emergency Management Agency Report FEMA 273, Washington, DC.

Bruneau, M. and Bhagwagar, T. (2002), “Seismic retrofit of flexible steel frames using thin infill panels”, *Eng. Struct*., **24**(4), 443-453.
CEN (1993), “EN 1993: Eurocode 3 - Design of steel structures”, European Committee for Standardization, Brussels.
CEN (2004), “EN 1998: Eurocode 8 - Earthquake resistance design of structures”, European Committee for Standardization, Brussels.
D’Ayala, D., Worth, J. and Riddle, O. (2009), “Realistic shear capacity assessment of infill frames: comparison of two numerical procedures”, *Eng. Struct*., **31**(8), 1745-1761.
Di Sarno, L. and Elnashai, A.S. (2009), “Bracing systems for seismic retrofitting of steel frames”, *J. Constr. Steel Res*., **65**(2), 452-465.
Gelfi, P. (2006), “SIMQKE-GR - Software for generating artificial accelerograms compatible with the response spectrum”, University of Brescia, Italy
Gunturi, S.K.V. and Shah, H.C. (1992), “Building specific damage estimation”, *Proceedings of the 10th WCEE ‘92*, Madrid, Spain, 6001-6006.
Madan, A., Reinhorn, A.M., Mander, J.B. and Valles, R.E. (1997), “Modeling of masonry infill panels for structural analysis”, *J. Struct. Eng*., **123**(10), 1295-1302.
Mohebkhah, A., Tasnimi, A. and Moghadam, H.A. (2008), “Nonlinear analysis of masonry-infilled steel frames with openings using discrete element method”, *J. Constr. Steel Res*., **64**(12), 1463-1472.
Mondal, G. and Jain, S.K. (2008), “Lateral stiffness of masonry infilled reinforced concrete (RC) frames with central opening”, *Earthq. Spectra*, **24**(3), 701-723.
OASP (2003), *Greek antiseismic code*, Greek Ministry of Environment and public works, Athens, Greece (In Greek).
Park, Y.J. and Ang, A.H.S. (1985), “Mechanistic seismic damage model for reinforced concrete”, *J. Struct. Eng. ASCE*, **111**(4), 722-739.
Paulay, T. and Priestley, M.J.N. (1992), *Seismic design of reinforced concrete and masonry buildings*, John Wiley & Sons Inc., Hoboken, NJ.
Reinhorn, A.M., Roh, H., Sivaslvan, M., Kunnath, S.K., Valles, R.E., Madan, A., Li, C., Lobo, R. and Park, Y.J. (2009), *IDARC 2D Version 7.0: A program for the inelastic damage analysis of structures*, Technical Report MCEER-09-0006, State University of New York, Buffalo, NY.
Sun, G., He, R., Qiang, G. and Fang, Y. (2011), “Cyclic behavior of partially-restrained steel frame with RC infill walls”, *J. Constr. Steel Res*., **67**(12), 1821-1834.
Tasnimi, A.A. and Mohebkhah, A. (2011), “Investigation on the behavior of brick-infilled steel frames with openings, experimental and analytical approaches”, *Eng. Struct*., **33**(3), 968-980.
Zarnic, R. and Gostic, S. (1997), “Masonry infilled frames as an effective structural subassemblage”, *Proceedings of the International Workshop on Seismic Design Methodologies for the next generation of codes '97*, Bled, Slovenia, A.A. Balkema, Rotterdam, 335-346

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