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SIMPLIFIED SEISMIC ASSESSMENT OF INFILLED REINFORCED CONCRETE FRAME STRUCTURES

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Abstract

The seismic assessment of structural performance due to ground shaking typically involves an analysis of several limit states. As such, the ability to accurately quantify the exceedance of any structural demand-based performance level is of great interest. Fragility curves are generally employed and are the end-result of an extensive numerical analysis campaign, which requires large computational effort and time depending on the complexity of the numerical modelling and the extent of the analysis procedure. In an effort to reduce the computational burden, SPO2IDA has been developed as a convenient tool for the assessment of structures. The tool empirically relates the results of static pushover analysis (SPO) to incremental dynamic analysis (IDA) via a library of empirical fitting coefficients for the different branches of the idealized SPO backbone. It can therefore characterize structural performance right up to structural collapse as a function of seismic intensity in a simple and intuitive manner.

SPO2IDA was initially developed for ductile reinforced concrete (RC) structures and steel moment-resisting frames; these typologies can be sufficiently represented by a SPO backbone with a bilinear yielding with a post-yield hardening accompanied by a post-capping strength degradation. This has resulted in the tool being widely adopted in seismic assessment guidelines. However, the same kind of structural behavior is generally not observed with the addition of masonry infills to RC frames - a prevailing structural typology that still requires attention and further study in the field of earthquake engineering. Such differences in backbone behavior compared to typical structures render the extension of the original SPO2IDA tool inappropriate and at times unconservative. To this end, the present study describes the recent extension of this simplified methodology for typologies with a backbone behavior more typical of infilled RC frames using single-degree of freedom oscillators. An extensive comparison of the existing models and the proposed approach is conducted to investigate the behavior and trends through numerical analyses when pushing infilled RC frames up to complete structural collapse. The newly-developed framework is represented through R–μ–T relationships representative of the dynamic behavior of the typology under scrutiny. The extended version of SPO2IDA for infilled RC frames is then verified and validated in terms of the matching of the produced IDA traces using an existing case-study building.

Keywords: Seismic Assessment, Collapse, Seismic Risk, Infilled Frames, Extended SPO2IDA

1. Introduction

Assessing the structural performance of existing reinforced concrete (RC) frame buildings designed prior to the introduction of modern seismic design provisions typically involves an analysis of serviceability limit states, collapse performance and the behavior of the structure itself conditioned on non-global collapse. However, the ability to accurately quantify the exceedance of any structural demand-based performance level requires extensive numerical modelling, ground-motion selection, non-linear dynamic analyses and post-processing of the subsequent results, which typically require large computational effort and time depending on the complexity of the numerical modelling and the extent of the analysis procedure adopted.

To somewhat reduce this computational burden, a number of simplified tools and methodologies for the assessment of RC frame structures have emerged in the literature over the past number of years, with the introduction of the SPO2IDA tool [1] being a notable example. It empirically relates the static pushover (SPO) curve to its corresponding incremental dynamic analysis (IDA) traces [2] in a simplified manner. The main scope behind such tools is to aid the quantification and mitigation of seismic risk with respect to different guidelines and procedures [3–7] and supply users with means with which to reduce computational effort and processing time.

These simplified tools employ what are typically referred to as R–μ–T relationships, developed through extensive dynamic analyses, which empirically relate the strength reduction factor, R, of a single degree of freedom (SDOF) oscillator to the elastic spectral demand of a ground motion to its period of vibration, T, and subsequent ductility demand, μ. Recent work [8] has shown that these existing tools may not be entirely adequate for the characterization of infilled RC frame typologies since the available empirical R–μ–T relationships were not established to represent such SPO backbone characteristics and mainly targeted other structural configurations, such as ductile moment frames designed with modern design code provisions. This is mainly because existing RC frames with masonry infill panels cannot be reasonably represented by a simple bilinear hysteretic backbone. In fact, studies by Dolsek and Fajfar [9,10], for example, were among the first to address this issue, both from a numerical modelling perspective and also a simplified implementation viewpoint.

Keeping the above remarks in mind, regarding the simplified assessment of RC frames with masonry infills, an extension of the existing SPO2IDA tool has been recently proposed by the authors [11] where an exhaustive and adequate assessment of these structural typologies was conducted. The aim of the newly-developed tool is to perform an assessment of the said typologies using SDOF oscillators and representative backbones in an efficient and simplified manner. To this end, the tool incorporates representative R–μ–T relationships established through extensive numerical campaigns on numerous infilled RC frames typologies as described in [11].

Therefore, this paper aims at characterizing the seismic response of infilled RC frames using SDOF oscillators, whose backbone is calibrated on numerous nonlinear static and dynamic investigations. Additionally, R-μ-T relationships representative of the typology are established and an extended version of SPO2IDA is introduced for carrying-out out a simplified seismic assessment of the concerned typology. A validation exercise using extended SPO2IDA is performed on existing case study building to introduce users to the tool developed herein.

2. Assessment of infilled RC frames with equivalent SDOF oscillators

2.1. Characterization of behavior

The response of infilled RC frames to the application of lateral loads can be outlined in a simplified manner through SPO analysis, where a structure is subjected to vertical gravity loads and a continuously increasing lateral load pattern until an ultimate displacement condition is reached. Figure 3 illustrates the differences in behavior between a bare frame, infilled frame and an infilled frame with a mechanism developed at a critical storey. The latter has been introduced in Nafeh *et al.* [11] following the characterization of numerous case-study frames as a fundamental aspect to truly comprehend and mimic the actual response of the said typology. As illustrated, the presence of masonry infills considerably modifies the lateral response when compared to the bare frame. That effect is noticeable through the increase in initial stiffness, peak lateral strength and sudden drop in lateral load-bearing capacity. The effect of the infilled frame's soft-storey mechanism is evident through their post-peak strength loss. Thus, the infill panels were removed to imply that a soft-storey mechanism has occurred due to the concentration of damage in one-storey. This allows the behavior of infilled RC frames once a mechanism has been formed to be mimicked. These differences are illustrated in Fig. 1, where the initial stiffness between the bare RC frame and the infilled frame with the critical storey's infill panels removed is noted. This implies that significant stiffness remains at the other non-critical storeys. Therefore, the response of the infilled RC frame should be inclusive of the infilled RC frame with the critical storey's masonry infill removed and not of the entirely bare frame as it more realistically represents the response of said frames.

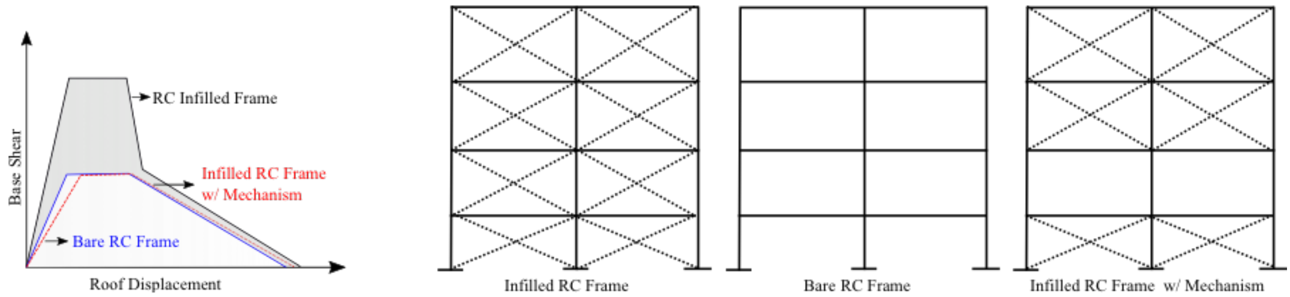
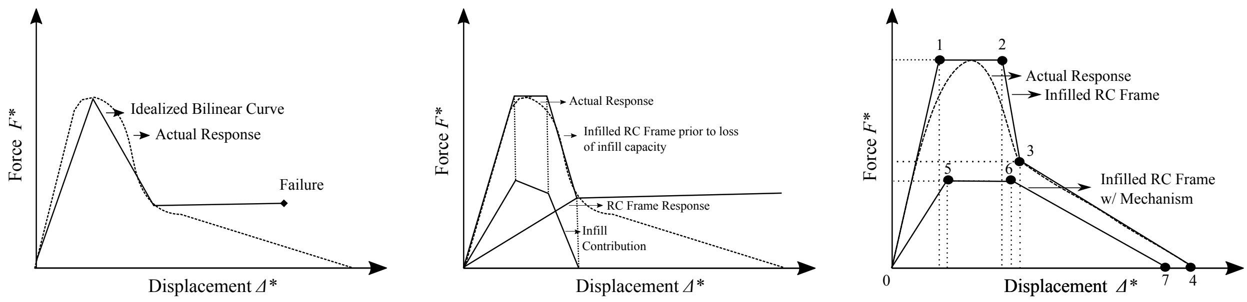


Fig. 1 - Differences in pushover curves between bare frame, completely infilled frame and infilled frame with mechanism developed at critical storey.

2.2. SDOF modelling

Section 2.1 examined the characterization of infilled RC frames via SPO analysis with reference to numerical models described in Nafeh *et al.* [11] that consider the individual structural element characteristics in a direct manner (i.e. non-linear static analyses). Nevertheless, when moving to the more computationally demanding dynamic analysis, the use of equivalent SDOF models may be a more suitable option (i.e. to capture key structural characteristics while permitting robust analyses in a quick and efficient manner). Examples of contexts that favor such an approach are present in simplified assessment methods such as [12] or [13]. In the context of infilled RC frames, this matter has received relatively little specific regard when compared to other prominent typologies. For example, D'Ayala *et al.* [14] published guidelines on how SDOF models may be developed for typologies including infilled RC frames where the response backbone is established using the results of a simple SPO analysis or by assuming standard lateral capacity ranges. It is then converted to an equivalent SDOF by assuming a first-mode dominant response discussed in detail in Nafeh *et al.* [11]. An ultimate displacement of the equivalent system is typically defined and the backbone response is thus assumed to follow the identified force-displacement relationship.

Fig. 2 illustrates approaches to capture the overall performance of infilled RC frames through SDOF oscillators. Nonetheless, particular aspects may still require improvements to better characterize the lateral response. For instance, considering a defined ultimate displacement capacity of these structures is of undoubted importance when examining their behavior up to collapse. To this end, Approach 1 may be more beneficial since Approach 2 does not consider a defined threshold for ductility capacity of the underlying RC frame. On the other hand, what is favorable about Approach 2 is the manner in which it segregates the masonry infill contribution from the RC frame contribution. Yet, this approach demands further attention since the subsequent response of the structure following the collapse of the masonry infill at the critical storey in the building does not exactly become the response of a completely bare frame, as highlighted in Section 2.1. O'Reilly and Sullivan [15] discussed this aspect briefly where the modal properties of the infilled RC frame's numerical model with the critical storey's infill removed showed a significant increase in the first mode period, but was still notably lower than that of the structure modelled without any masonry infill. Considering the above remarks, a more refined equivalent SDOF modelling approach for infilled frames is proposed by Nafeh *et al.* [11] and summarized herein where the actual response of the infilled RC frame is combined with the structural response once the critical storey has formed a mechanism and its infill panels have collapsed (Fig. 2c). The proposed model comprises a combination of this latter model and the infill contribution to give the 0–1–2–3–4 response initially but reduces to the 0–5–6–7 response once the infill has collapsed.



(a) (b) (c)

Fig. 2 - Illustration of (a) Approach 1 applied in past studies such as Villar-Vega *et al.* [13], (b) Approach 2 established by Dolsek and Fajfar [9] for infilled frames and (c) illustration of the recommended modelling strategy proposed in Nafeh *et al.* [11].

2.3. Evaluation of performance

A series of IDAs were performed to evaluate the suitability of the aforementioned equivalent SDOF models for dynamic analysis. The dynamic behavior of these models is compared to the response of the actual MDOF model of a 4-storey infilled RC frame structure. The FEMA P695 [5] far-field ground motion set was used and the spectral acceleration at the first mode period of vibration, $Sa(T_1)$ was selected as the intensity measure. IDA was conducted until the complete collapse of the structure, defined as reaching a 10% storey drift in any storey for a given record, as discussed by O'Reilly *et al.* [16]. The median IDA trace was computed from the individual ground motion at each intensity. The differences between Approaches 1, 2 and the proposed model are examined and compared to the response of the actual MDOF model, shown in Fig. 3. The strength ratio, R, defined as the ratio between the recorded and yield spectral accelerations is plotted against the ductility of the frames, μ.

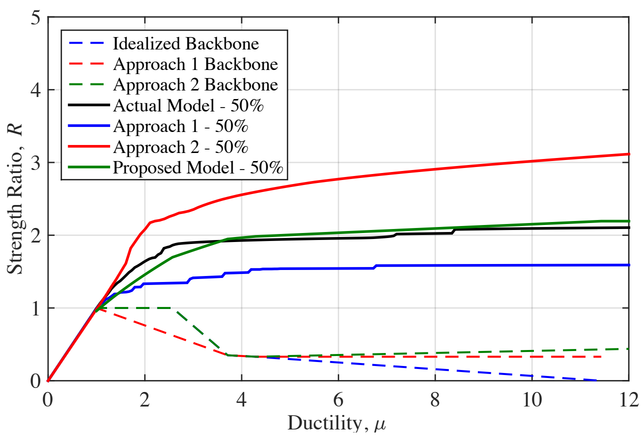


Fig. 3 - Comparison of the median IDA curves normalized as R–μ using Approaches 1 and 2, the actual MDOF models and the proposed approach for a 4-storey strong-infilled frame.

For a given intensity, Approach 1 appears to overestimate the dynamic response of the structure due to the model's oversimplification of the initial portion of the backbone curve shown in Fig. 2a. The use of a single peak point followed by a sudden drop in lateral load-bearing strength to model the effects of masonry infill tends to result in an increased flexibility (i.e. underestimation of initial stiffness) and subsequently larger displacement demand when compared to the actual model. Maintaining the resistance with increasing levels of intensity is due to the adoption of a bilinear backbone RC frame response with no definition of an ultimate ductility threshold (i.e. no failure point of the RC frame), illustrated in Approach 2 in Fig. 2b. Therefore, the SDOF oscillator is able to endure increased values of intensity without their IDA curves ever actually 'flat-lining' (i.e. an indication of structural collapse observed as a large increase in demand for small increasing values of intensity where the IDA curves flatten out) in the absence of a proper failure point definition. On the other hand, for the SDOF model proposed here in Fig. 2c, the median values are much more aligned with the median response of the actual model, especially the intensity at the collapse of the structures, demonstrating its suitability. As such, given the significant differences observed in the collapse intensity estimation among the approaches examined, it is believed that the numerical modelling approach suggested here for infilled RC frames should be the one adopted in studies of this sort.

3. Simplified assessment of infilled RC frames

3.1. Existing methods

The previous section analyzed different methods to characterize the seismic response of infilled RC frames using SDOF oscillators along with dynamic analysis with ground motion records. The subsequent sections, however, discuss the suitability of using a series of closed-form solutions to relate the SPO backbone to its dynamic response via ground motion excitation. These methods are commonly referred to as R–μ–T relationships as they provide the relationship between the strength ratio, R, the ductility, μ, and the period, T, which is usually defined as the first mode period, T_1.

There have been numerous studies on such relationships for different structural systems over the years [17–20]. Of particular pertinence to this study is the aforementioned work by Dolsek and Fajfar [9], who utilized the modelling approach described in Figure 3b to quantify these relationships for infilled RC frames. However, these relationships inherently contain some limitations described in the equivalent SDOF modelling approach described in Section 2.2 and 2.3. Additionally, the relationships provided in these studies described the mean response without addressing the associated variability. This aspect were indirectly addressed in the development of the SPO2IDA tool [1] whereby the relationship between the SPO curve and its corresponding IDA percentiles (i.e. 16%, 50% or median, and 84%) was quantified in an empirical manner. SPO2IDA represents an attractive method to estimate the seismic demand and capacity of first-mode-dominated MDOF systems in regions ranging from near-elastic to global collapse. It has been primarily developed to analyze bilinear systems with some form of degradation, which corresponds well to the behavior of ductile moment frames, for example. However, in the case of existing RC frames, these tend to have masonry infills that significantly alter the structural behavior with the increase of strength and stiffness, as explained in Section 2.1. This modified behavior is incompatible with tools such as SPO2IDA as they were developed with R–μ–T relationships more suited to ductile systems typical of new construction. The following section describes the extension of the SPO2IDA tool developed by Nafeh *et al.* [11] that overcomes the addressed limitations discussed herein through incorporating R–μ–T relationships representative of this structural typology.

3.2. Newly developed method

The previous sections highlighted the considerations to be accounted for in order to incorporate the behavior of infilled RC frames in the extension of the SPO2IDA tool outlined in [11]. To this end, characteristic R–μ–T relationships of the said typology were developed using a large sample of representative SDOF systems (Fig. 2c) analyzed using IDA to characterize their performance with increasing intensity up to collapse and allow the fitting of a new empirical library of R–μ–T relationships. Two-step regression was employed to establish and quantify the relationship between the dynamic strength ratio, R, and the ductility, μ, for a given SPO backbone specific to infilled RC frames and its period, denoted T^*. The framework for extension is discussed in detail in Nafeh *et al.* [11]. This tool has also been implemented both in a MS Excel Spreadsheet and a MATLAB script, available at https://github.com/gerardjoreilly/InfilledRC-SPO2IDA.

The extended SPO2IDA algorithm to determine the relationship between the roof displacement of the structure, Δ, and the intensity required to reach that demand, $Sa(T_1)$, can be outlined as follows:

1. Following a SPO analysis and backbone linearization and for any ductility, μ, in a given branch of the SPO curve as shown in Figure 4, the roof displacement of the MDOF, Δ, is found using:

$$\Delta = \Gamma . \mu . \Delta_y^* \tag{1}$$

where Γ is the MDOF-SDOF transformation factor obtained from eigenvalue analyses, Δ_y^* is the SDOF displacement at yield.

2. The yield spectral acceleration, Sa_y, is found (Equation 7a-b)
3. Using the coefficients fitted for the x% fractile of the IDA for a certain branch of the IDA, the R_{dyn} is found and then converted to a spectral acceleration $Sa(T_1)$ using:

$$Sa(T_1) = R_{dyn} Sa_y \Gamma \tag{3}$$

This process is repeated for each of the branches shown in Fig. 4, for each of the percentiles required (e.g. x = 16%, 50% and 84%) to get an IDA curve similar to that illustrated in Figure 4.

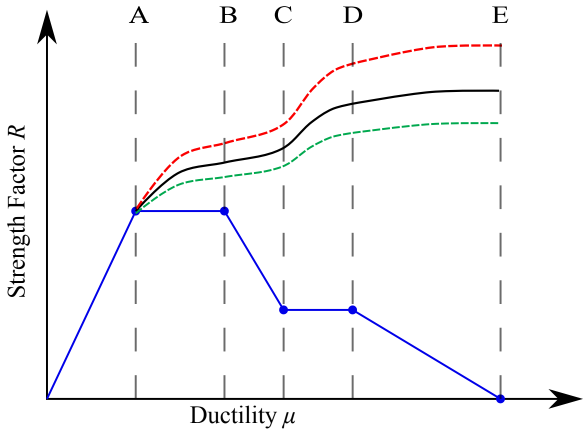


Fig. 4 - Idealised strength ratio, R, versus ductility, μ, of a given infilled RC frame's equivalent SDOF.

3.3. Evaluation of performance

To evaluate the performance of the outlined simplified methods, both were implemented here and compared with the results of the IDA conducted in Section 2.3 for the MDOF structures. In other words, the SPO curves were characterized as shown in Fig. 2, then the idealized backbones for the Dolsek and Fajfar [9] model (Approach 2 in Fig. 2) were fitted. To quantify the spectral acceleration required to reach a given level of ductility, the R–μ–T relationships in Dolsek and Fajfar [9] were utilized. The SPO2IDA tool was also implemented. Hence, the efficiency of these methods in quantifying the structural response to the point of collapse is assessed. This comparison differs to that of Section 2.3 in that no dynamic analyses were performed on the SDOF models and only R–μ–T relationships were utilized.

For the approach outlined by [9], a series of empirical relationships were used for quantifying a dynamic R–μ–T relationship for the mean response. For the case of SPO2IDA, two alternative approaches are permitted when carrying out the equivalent SDOF conversion. These relate to using the transformation factor, Γ, or by simply using the ratio between the yield force and the total weight of the system, C_0. The first option, corresponding to an eigenvalue analysis is used herein.

When compared to the median response of the actual model, Fig. 5 shows that both SPO2IDA and Approach 2 tend to a reasonable match with the MDOF model curve for low levels of ductility but once the response exceeds the softening branch (i.e. first negative stiffness branch), they both begin to largely overestimate the strength ratio. In the case of Approach 2, this arises due to the assumption of infinite RC frame ductility, meaning that the SDOF oscillator possesses no physical means of losing its lateral capacity and can continue withstanding increased ground shaking. In the case of SPO2IDA, this overestimation is due to the limitation of the backbone parameters described in Nafeh *et al.* [11] whereby a constant residual strength branch is considered instead of a gradually degrading one (i.e. between points D and E of Figure 4). Fig. 5 shows that SPO2IDA clearly expresses an overestimation of the capacity since it has not been developed or adapted for these specific typologies, hence missing some of the key performance aspects towards collapse and post-peak branches. This is noted to be consistent with the initial findings of O'Reilly and Sullivan [21] who reported that, when applied to infilled RC frames, SPO2IDA tended to largely overestimate the median collapse intensity when compared to IDA results, potentially leading to an unconservative prediction of the performance.

Subsequently, the adequacy of the tool was evaluated by comparing the IDA traces for the 4-storey strong infilled frames described in O'Reilly and Sullivan [21] with the prediction of the extended SPO2IDA. Fig. 5 shows this for the 16th, 50th and 84th percentiles obtained from SPO2IDA and extended SPO2IDA, where the plots are normalized to strength ratio and ductility and compared with results from incremental dynamic analysis. The IDA curves obtained from the extended SPO2IDA match well with the trends and the general range of strength demand exhibited by the traditional IDA of the actual models as opposed to an overestimation in capacity from both SPO2IDA and the Dolsek & Fajfar methodology. Fig. 5b clearly illustrates an improvement in the predictive ability of extended SPO2IDA in estimating the dynamic capacity of the structural system to increasing levels of intensity. Changes in the peak deformation or ductility patterns are preserved when comparing both to along with onset of stiffness, hardening and the other branches of the idealized backbone curve in Fig. 6, up until the strength degradation and eventual collapse.

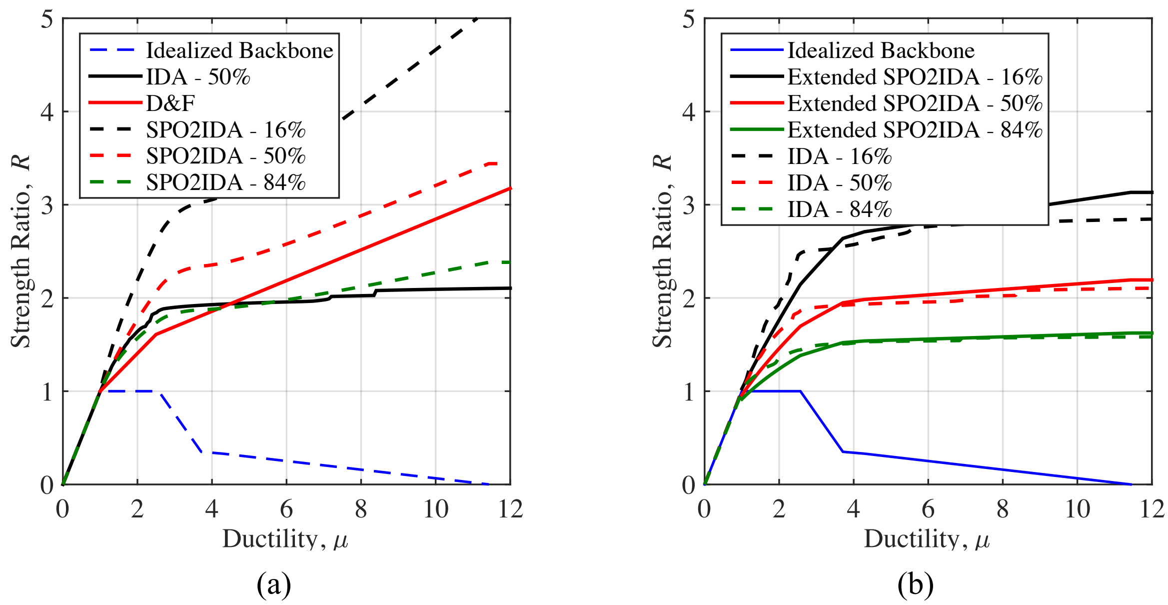


Fig. 5: Comparison of (a) the SPO2IDA R–μ fractiles (16%, 50% and 84%) response using IDA and Dolsek and Fajar [9] and (b) extended SPO2IDA on the actual MDOF for 4-storey strong infilled frame.

4. Validation of extended SPO2IDA using existing school building in Italy

The building chosen to further validate the results obtained using the extended SPO2IDA was a 3-storey RC school building with masonry infills located in Central Italy. The school building considered for the validation herein was constructed in the 1960s before the introduction of seismic design provisions and whose numerical modelling assumptions are examined in detailed by O'Reilly *et al.* [22,23] and illustrated in Fig. 6.

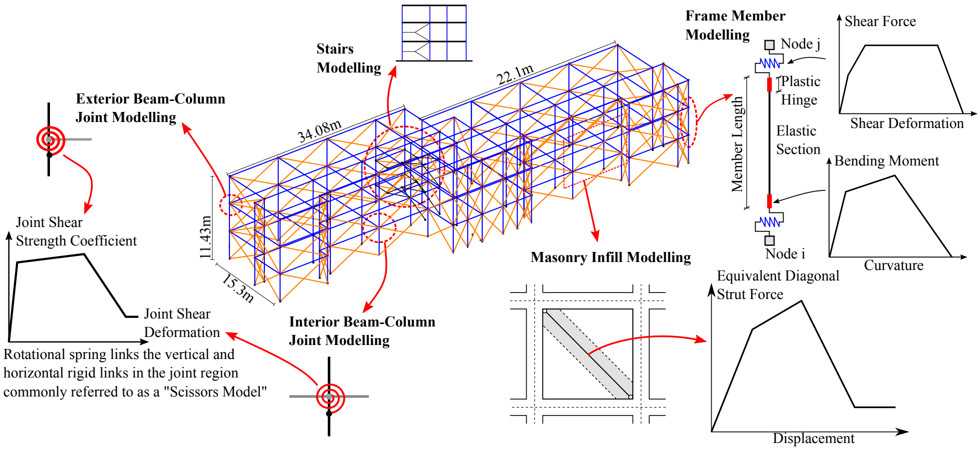


Fig. 6: Layout and numerical modelling assumptions of the case study school building used for validation of the proposed tool. Adapted from [22]

To utilize extendedSPO2IDA, (a) eigenvalue and (b) static pushover analyses are performed, followed by (c) an equivalent linearization of the input SPO results using the control points shown in Fig. 4 (points corresponding to yielding, hardening, softening, residual plateau and ultimate capacity). The tool then computes the modal properties and normalizes the pushover curve to strength ratio-ductility then interpolates using closed-form solutions the 16th, 50th and 84th percentiles IDA traces. The output can be subsequently utilized to construct risk metrics such as fragility functions at any given limit state. To this end, SPO analysis was performed to quantify the building's response in both directions and the results are illustrated in Fig. 7. The highlighted points present the fitted linearization to the SPO curves previously described in Fig. 2c, which are needed as input to the extended SPO2IDA. The fundamental period of vibration, corresponding to both x and y directions ($T_{1,x}$ & $T_{2,x}$) were determined from modal analysis. Additional parameters required for the simplified analysis are the individual floor masses m_i and the fundamental mode shape Φ, also easily obtained from eigenvalue analysis. The yield spectral acceleration, Sa_y, is computed using equation 2 above. Extended SPO2IDA then computes the MDOF-SDOF parameters summarized in Table 1 as follows:

$$m^*{}_x = \sum_i m_{i_x}\phi_{i_x} = (985 * 0.218 + 985 * 0.556 + 806 * 1) = 1554.49\ tonnes \quad (4a)$$

$$m^*{}_y = \sum_i m_{i_y}\phi_{i_y} = (985 * 0.222 + 985 * 0.571 + 806 * 1) = 1572.83\ tonnes \quad (4b)$$

$$\Gamma_x = \frac{m^*{}_x}{\sum_i m_{i_x}{\phi_{i_x}}^2} = \frac{1554.49}{1149.58} = 1.35 \quad (5a)$$

$$\Gamma_y = \frac{m^*{}_y}{\sum_i m_{i_y}{\phi_{i_y}}^2} = \frac{1572.83}{1167.54} = 1.35 \quad (5a)$$

$$\Delta^*_{y,x} = \frac{\Delta_{y,x}}{\Gamma_x} = \frac{0.015}{1.35} = 0.011 \quad (6a)$$

$$\Delta^*_{y,y} = \frac{\Delta_{y,y}}{\Gamma_y} = \frac{0.015}{1.35} = 0.011 \quad (6a)$$

$$Sa_{y,x} = \frac{4\pi^2 \Delta^*_{y,x}}{{T_x}^{*2}} = 4\pi^2 * \frac{0.011}{0.621^2 * 9.81} = 0.116\ g \quad (7a)$$

$$Sa_{y,y} = \frac{4\pi^2 \Delta^*_{y,x}}{{T_x}^{*2}} = 4\pi^2 * \frac{0.011}{0.365^2 * 9.81} = 0.336\ g \quad (7b)$$

Multiple stripe analysis was conducted to assess the dynamic response of the structure with increasing levels of intensity at different return periods using different sets of hazard-consistent ground motion sets selected for a site in Central Italy, which are described in more detail in O'Reilly *et al.* [22]. The parameters needed to perform the MDOF to SDOF conversion in the extended SPO2IDA were determined from eigenvalue analysis, and are detailed in Table 2, following the equivalent SDOF conversion steps detailed in Nafeh *et al.* [11]. Using the MSA results for this school building, the performance of the school was quantified at a number of intensity levels, with some collapse cases noted for higher return period. The median collapse intensity and the dispersion due to record-to-record variability were determined to be 1.63 g and 0.37, respectively.

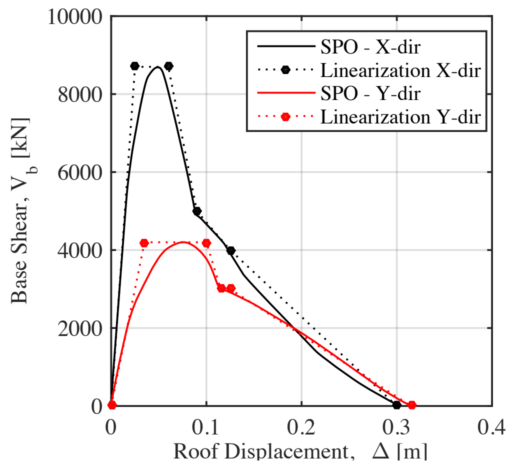
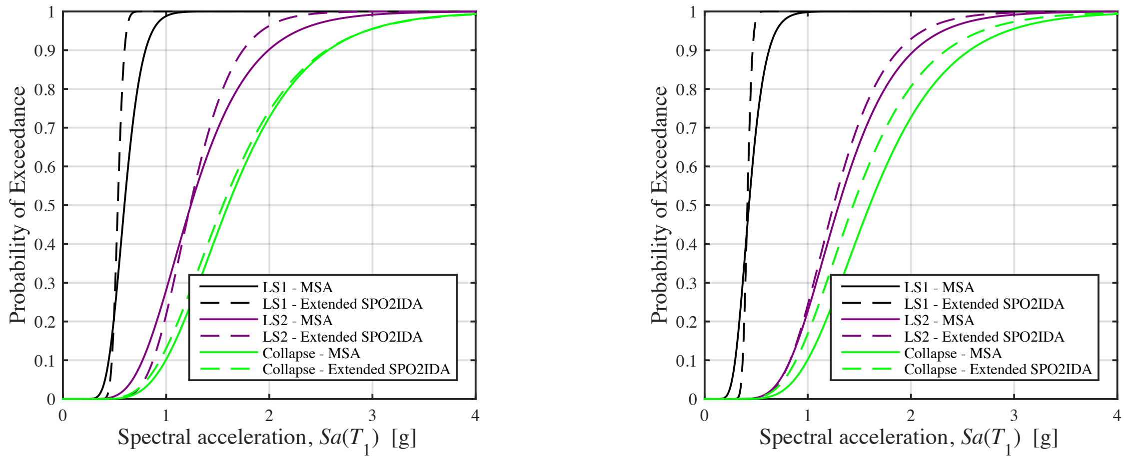


Fig. 7: Static pushover curves of the case study building in both principal directions with the linearized proposed fitting.

Table 1. MDOF-to-SDOF conversion parameters from eigenvalue analysis

X-direction				**Y-direction**			
T_1 **[in s]**	m^* **[in kg]**	Γ	Sa_y **[in g]**	T_1 **[in s]**	m^* **[in kg]**	Γ	Sa_y **[in g]**
0.365	1572.83	1.35	0.116	0.621	1554.49	1.35	0.336

Furthermore, two additional limit states corresponding to the peak resistance and the end of the residual plateau (point 3 in Fig. 2c) were defined in order to compare the extended SPO2IDA and the MSA results. Roof displacements corresponding to these limit states (LS1 and LS2 herein) were identified in both principal directions of the school building and their exceedance with respect to increasing intensity was established from the MSA results. Similar to the collapse cases above, fragility functions were fitted and are reported in Fig. 8 for both directions. Using the SPO curves shown in Figure 8 and the modal parameters described in Table 2 for both directions, the 16%, median and 84% IDA percentiles were established until collapse via the extended SPO2IDA. Using these traces, the associated fragility function for each limit state previously described could be established. These were identified and are also plotted in Fig. 8. Comparing the fragility functions, a good match is observed between the two sets in both directions of the building. This is true both in terms of the median intensity required to exceed each of these performance limit states and also the level of dispersion.



(a) (b)

Fig. 8: Comparison of the fragility functions derived for three limit states including the collapse of the school building obtained using multiple stripe analysis and the extended SPO2IDA tool

5. Summary and conclusions

Reinforced concrete (RC) infilled frames constitute a high percentage of the building stock in Italy, the Southern Mediterranean and other parts of the world. Therefore, the seismic assessment of the structural performance of infilled RC structures remains an important subject in earthquake engineering. When evaluating the associated risk with such structures, the ability to quantify their behavior with respect to increasing levels of seismic intensity is a critical aspect. Non-linear dynamic analyses procedures like incremental dynamic analysis (IDA) are computationally-demanding procedures in terms of the large effort and time required when performed on detailed numerical models built to capture the potential failures models common in these structural typologies. This study has reviewed simplified approaches to assess structures. These approaches comprised both equivalent single degree of freedom (SDOF) oscillators and their definition and empirical relationships, referred to as R–μ–T relationships. As a result of this study, the following can be concluded:

- The over-simplified nature of existing approaches in characterizing the behavior of infilled RC structures using equivalent SDOFs is not entirely representative when compared to the response of detailed numerical models;
- A more refined equivalent SDOF modelling approach was subsequently proposed; it incorporates all aspects of the global behavior of such structures when subjected to lateral loads and performs better when compared to the existing models;
- The applicability of the tool relating the static pushover (SPO) curve of a structure to its IDA percentiles (SPO2IDA) was reviewed and seen not to be applicable to RC frames with masonry infills. This was seen to be due to the incompatibility of the initial assumptions made regarding SPO backbone branches in the original tool that do not fit well with the specific characteristics of infilled RC frames;
- The recently-developed extended SPO2IDA tool enables the user to perform effortless dynamic analyses through empirical relationships established mainly for the typology under scrutiny.
- Extended SPO2IDA was then verified through the comparison of IDA traces and with the results of an independent study on an existing school building in Central Italy of the same typology. These results validated the applicability of this simplified tool for the collapse assessment and general characterization of their structural response.

6. Acknowledgements

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