# Distinction Between No and Slight Damage States for Existing RC Buildings using a Displacement-Based Approach

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**SUMMARY**
The purpose of this study is to indicate the initiation of damage (and thus loss) of reinforced concrete (RC) structures for displacement-based assessment. In order to define this point it is necessary to investigate the behaviour of non-structural elements, namely infill walls. Throughout this study 20 infilled RC frames are analysed which are extracted from existing and poorly designed/constructed buildings from the European-Mediterranean Region. By utilizing advanced, distributed-plasticity nonlinear analysis software, SeismoStruct [SeismoSoft, 2010], a Displacement-Based Adaptive pushover (DAP) type of analysis has been conducted for all case study frames to obtain the capacity curves for bare and infilled cases of each frame. The aim is to obtain a simple relationship between the predefined first limit state of bare frames (LS1) and the limit state where infill walls exhibit cracks which would require repair and/or strengthening. The proposed formula for that relationship is aimed to be straightforward, easy to be applied and implemented in earthquake loss assessment studies.

*Keywords: Infilled RC frames; limit states; loss assessment; displacement based adaptive pushover (DAP)*

## 1. INTRODUCTION

Although they are not included in structural design, the contribution of infill walls must be somehow taken into account in assessment studies, since the presence or lack of infill walls may change the overall result substantially, especially in structures where the RC frame is under-designed and not sufficient in terms of strength and ductility.

The infill walls may show favourable as well as adverse effects on the seismic response of a structure (Fardis and Panagiotakos, 1996). The infilled frame can be superior to the bare frame in many aspects such as positive contribution to in-plane strength and stiffness of overall strength and improved energy dissipation, but only if they are properly designed and placed throughout the structure. On the other hand, improperly designed and unevenly distributed infill walls may cause several adverse effects such as short column phenomena, soft story, torsion or out-of-plane failure. The infill walls may be the major reason of financial losses in cases where the structure is not moderately or substantially damaged. Failure of infill walls requires serious repair and cosmetic works. A recent study by Bal et al. [2007] and [2008b] has shown that for the Turkish building stock, for example, the ratio of repairing a slightly damaged building to reconstructing that building may be up to 16%. The slight damage referred to herein is the damage mostly to the non-structural elements but it may also include some very minor structural damage in some cases.

This paper focuses on the influence of infill walls on the stiffness and displacement capacity of RC frames, and aims to identify at which displacement/drift damage these elements begin experiencing damage that requires repair. This will allow an initial damage state (between no and slight damage) to be identified and used within a displacement-based assessment procedure.

## 2. DISPLACEMENT-BASED LOSS ASSESSMENT (DBELA) METHODOLOGY

Due to the need for a better methodology in terms of good correlation, transparency, theoretical robustness, computational feasibility and a full probabilistic treatment of the variables for the loss assessment of RC buildings, a displacement-based methodology has been proposed by Pinho et al. [2002] and by Glaister and Pinho [2003]. The methodology has been developed for practical uses for RC buildings (Crowley et al., 2004) and as a crucial element of the method, the period-height relationship for infilled emergent-beam frames has also been recently introduced by Crowley and Pinho [2006]. Furthermore, a probabilistic framework by Crowley et al. [2006a] is also included in the original method to account for the uncertainty in the displacement demand and capacity of the exposed buildings. With the help of this framework, the behaviour under the defined demand can be estimated using sets of equations and assumptions depending on the classification of the building geometry, bearing system and material properties. For further information, readers are advised to consult the publications of Bal [2008] and Bal et al. [2010].

Infilled frames as compared to bare ones result in an increase in the strength and slight decrease in the displacement capacity. The former, which has the effect of increasing the limit state stiffness and decreasing the limit state period, has already been considered in the calculation of the limit state periods (Bal, 2008). Thus, for DBELA calculations the slight decrease in the displacement capacity for the infilled frames has to be defined. Through the probabilistic framework of the DBELA approach, a decrease in the displacement capacity can be included in the calculations by introducing an additional parameter, $\beta_i$. This parameter is expressed as a statistical factor, represented with a mean value, standard deviation and distribution.

The displacement capacity equations of bare RC frames are presented by the following equations; see Equation 2.1 and 2.2, in the study of Bal [2008]:

$$\Delta_{SLSi} = \left[0.5\delta_e H_T \varepsilon_y \frac{l_b}{h_b}\right]\beta_1 + \left[0.5\left(\varepsilon_{C(LSi)} + \varepsilon_{S(LSi)} - 1.7\varepsilon_y\right)\delta_e H_T\right]\beta_2 \tag{2.1}$$

$$\Delta_{SLSi} = \left[0.43\delta_e H_T \varepsilon_y \frac{h_s}{h_c}\right]\beta_1 + \left[0.5\left(\varepsilon_{C(LSi)} + \varepsilon_{S(LSi)} - 2.14\varepsilon_y\right)h_s\right]\beta_2 \tag{2.2}$$

In this study, by conducting displacement-based adaptive pushover analyses [Antoniou and Pinho, 2004] the decrease in the limit state displacement capacity due to the presence of masonry infill walls is determined for both embedded- and emergent-beam frames whose descriptions can be found in Bal [2008]. Following the results of the analyses conducted within the scope of this study, tentative average values for $\beta_1$ are presented, noting that the original values of $\beta_2$ presented in the study of Bal [2008] are still valid.

## 3. CASE STUDIES AND MODELLING ASSUMPTIONS

### 3.1. Case Studies

The infilled frames used in this study are selected not only from the different parts of Europe and Turkey but also from the different construction eras in order to cover a reasonable part of the existing, under-designed mid-rise RC building stock in the region. The geometrical and structural properties of the frames also show quite a significant variety in terms of number of storeys, number of bays, element cross sections, orientation of columns, material properties, etc. Moreover, among the 20 frames, 14 of them consist of conventional emergent beams whereas 6 of them have embedded shallow beams, a common construction practice in Europe, which also aimed to help to increase the variety of the models. Hence, the case study frames can be divided into 2 main sets as infilled frames with emergent beams and infilled frames with embedded beams (Table 3.1).

**Table 3.1.** Case studies used in analyses

| Emergent Beams | | | Embedded Beams | | |
|---|---|---|---|---|---|
| **Frame** | **Number of Stories** | **Material Characteristics (MPa)** | **Frame** | **Number of Stories** | **Material Characteristics (MPa)** |
| PFN-2-1 | 2-storey | fc=22, fy=230 | Italian_70s_3Str | 3-storey | fc=20.75, fy=380 |
| Pavia_3Str | 3-storey | fc=17, fy=370 | Portuguese_60s_4Str | 4-storey | fc=24, fy=337 |
| ICONS_4Str | 4-storey | fc=16.3, fy=344 | Romanian_30s_5Str | 5-storey | fc=15, fy=500 |
| PFN_4_1 | 4-storey | fc=16.8, fy=400 | Italian_6Str | 6-storey | fc=19.6, fy=273 |
| PFN_4_2 | 4-storey | fc=16.7, fy=371 | Turkish_7Str | 7-storey | fc=16.7, fy=371 |
| Veli_5Str | 5-storey | fc=16.7, fy=371 | Italian_7Str | 7-storey | fc=22, fy=230 |
| Italian_6Str | 6-storey | fc=19.6, fy=273 | | | |
| PFN_6_1 | 6-storey | fc=16.8, fy=400 | | | |
| Project_5a_1 | 6-storey | fc=10, fy=371 | | | |
| Ferracuti_6Str | 6-storey | fc=33, fy=414 | | | |
| PFN_6_2 | 6-storey | fc=16, fy=371 | | | |
| Yugoslavian_7Str | 7-storey | fc=29, fy=375 | | | |
| PFN_7_1 | 7-storey | fc=16, fy=371 | | | |
| PFN_8_2 | 8-storey | fc=16.7, fy=371 | | | |

## 3.2. Modelling Assumptions

To perform the analyses of the frames, the advanced, distributed-plasticity nonlinear analysis software, SeismoStruct is used by conducting Displacement-Based Adaptive Pushover (DAP) type of analyses, as proposed by Antoniou and Pinho [2004] and by Pinho [2005]. The displacement-based adaptive pushover method is a powerful analysis tool for the deformation based nonlinear static analysis. A displacement pattern, which is continuously updated during analysis in accordance with the stiffness state at every step, is applied to the structure yielding better response estimations as compared to force-based counterparts. Besides these aspects, the DAP method is more preferable for the analysis of infilled frames due to their rather intrinsic constitutive relationship since it shows a stable behaviour in all types of analysis [Antoniou and Pinho, 2004].

In the modelling of RC elements, fibre-based element modelling, i.e. distributed inelasticity is preferable to concentrated inelasticity modelling especially where assessment of the structures is concerned. This is due to the fact that by using distributed inelasticity elements, the empirical response parameters do not need to be calibrated against the response of the actual or ideal frame element under idealized loading conditions. Moreover, from the point of view of assessment of a structure, selection of fibre-based elements is more favourable owing to its ease of accessibility to the sectional strain values which are used to identify damage levels, and limit states, which are defined based on the exceedance (or non-exceedance) of strain value at elements.

The frame elements are modelled in SeismoStruct with force-based (FB) elements by transforming the existing models with displacement-based (DB) elements. This action is performed due to the well-known subjectivity of the DB elements to sectional response (Calabrese et al., 2010). The FB formulation can be always regarded as exact regardless of the level of inelasticity since it does not depend on the assumed sectional constitutive behaviour and never restrains the displacement field of the element. However, to model the actual reinforcement patterns along the element, an exception is applied to the non- discretisation rule. In that case, when the element has different rebar configuration, it is not only divided into 2 or 3 elements but also 2-3 integration sections are assigned for FB

formulation. Otherwise convergence difficulties would be probable (Calabrese et al., 2010).
To model the nonlinear response of infill walls, a four node masonry panel element defined by six strut members, where four of them are for axial loads and two of them for shear loads, is used which was initially programmed by Crisafulli [1997] and verified by Smyrou *et al*. [2006], Smyrou [2006] and Smyrou *et al*. [2011], see Figure 3.1.

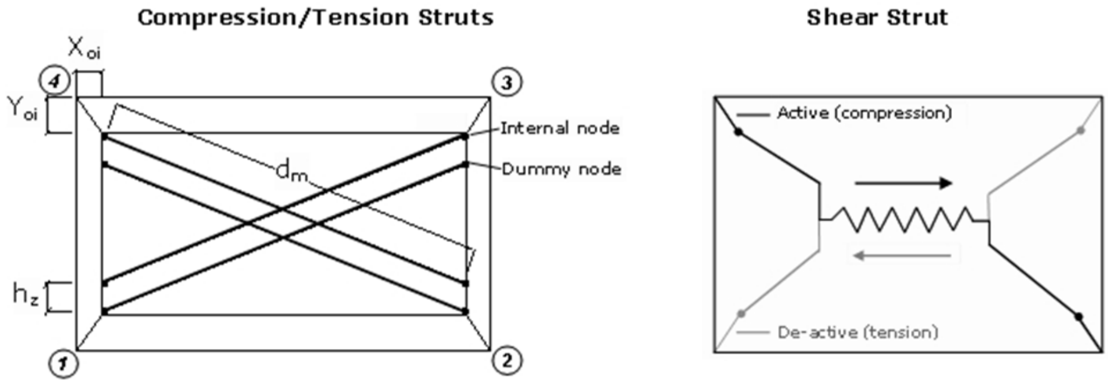



**Figure 3.1.** Infill panel element implemented in SeismoStruct (Crisafulli, 1997)

Several parameters including elastic Young's modulus, $E_m$, compressive strength of the diagonal struts, $f_{m\theta}$, related strain values, empirical factors and geometrical attributes have to be defined in SeismoStruct to implement the masonry panel element. Details of these values can be found in the study of Özcebe [2011].

Along with the parameters explained above, a parametric study is established to assign $f_{m\theta}$ and $\varepsilon_m$ values for the infilled walls where there is no information available. A normal distribution for these parameters is assumed and random values are generated in the limits of 1.9-3.2 MPa ($\mu$=2.55MPa, $\sigma$=0.22MPa) and 0.0002-0.0018 ($\mu$ =0.001, $\sigma$ =0.000267) for $f_{m\theta}$ and $\varepsilon_m$, respectively. It should be also noted that aforementioned limit values of $f_{m\theta}$ and $\varepsilon_m$ are extracted from a range of mean plus/minus three standard deviations. The obtained values are sorted and assumed correlated such that the lower the compressive strength of the infill, $f_{m\theta}$, the smaller the strain at maximum stress, $\varepsilon_m$. All $f_{m\theta}$ and $\varepsilon_m$ pairs assigned to the case studies are presented in Table 3.2.

**Table 3.2.** $f_{m\theta}$ and $\varepsilon_m$ values used in case studies

| Case Study | $f_{m\theta}$ | $\varepsilon_m$ | Case Study | $f_{m\theta}$ | $\varepsilon_m$ |
|---|---|---|---|---|---|
| *PFN-2-1 | 2248 | 0.000647 | *PFN_6_2 | 2254 | 0.000785 |
| Pavia_3Str | 2550 | 0.001 | Yugoslavian_7Str | 2550 | 0.001 |
| ICONS_4Str | 2850 | 0.001 | *PFN_7_1 | 2043 | 0.000615 |
| *PFN_4_1 | 2554 | 0.000961 | *PFN_8_2 | 2716 | 0.001163 |
| *PFN_4_2 | 2876 | 0.001433 | *Italian_70s_3Str | 2599 | 0.001017 |
| Veli_5Str | 2550 | 0.001 | *Portuguese_60s_4Str | 2212 | 0.000625 |
| Italian_6Str | 2550 | 0.001 | *Romanian_30s_5Str | 2840 | 0.001167 |
| *PFN_6_1 | 2365 | 0.000877 | *Italian_6Str | 2284 | 0.000824 |
| *Project_5a_1 | 2962 | 0.001467 | Turkish_7Str | 2550 | 0.001 |
| *Ferracuti_6Str | 2504 | 0.000922 | *Italian_7Str | 2633 | 0.001024 |

* Case studies in which the values are assigned from the parametric study.

## 4. LIMIT STATES AND RESPONSE OF THE FRAMES

In this study, the aim is to find a relation for the identification of the initial damage states (that will occur before structural yielding and be concentrated in the non-structural elements) in the displacement-based analysis of existing RC structures. This identification is required for a more accurate estimation of the financial cost of damage.

Certain structural material strain values are defined for the first limit state, LS1 for both concrete and reinforcement which are $\varepsilon_{core}$=-0.0045 and $\varepsilon_s$=0.013, respectively (Bal et al., 2008a and Bal, 2008). Moreover, three limit state strains for infills are also defined such as strain at maximum base shear, strain at maximum stress, $\varepsilon_m$ and ultimate strain, $\varepsilon_u$. The last two strain values are also selected from a recent study about the properties of the existing RC building stock, which is given in the relevant publications by Bal *et al*. [2007] and [2008b]. The ultimate strain, $\varepsilon_u$ is selected as 0.003 as the mid-value of the values recommended by the aforementioned references. Three other limit states associated with the infill walls are defined as LS0_1, LS0_2 and LS0_3 corresponding to the infill strain value at maximum base shear, the strain value at maximum stress, $\varepsilon_m$ and ultimate strain, $\varepsilon_u$, respectively.

All frames are analysed twice as an infilled frame and a bare frame by utilizing DAP analysis. The capacity curves for both types of analyses are obtained by indicating the limit states defined above, namely LS1 (for infilled and bare frames), LS0_1, LS0_2 and LS0_3 for infilled frames (Figure 4.1). Therefore, for both structural and non-structural elements, the required points are shown on the pushover curves.

The discussion and observations based on the capacity curves are such that:

- A typical trend of infilled frames is clearly seen in Figure 4.1. A high level of base shear at relatively low level of displacements is followed by a drastic drop in strength.
- The first limit state of the infilled frame, LS1infilled is always achieved before that of the bare frame. Their relations are also presented in Table 4.1 with the statistical interpretations. This is something expected since the infills firstly increase the stiffness and strength by decreasing the deformation capacity, and secondly because of the concentration of force on beam-column joints due to the infill struts.
- The applied displacement patterns for DAP analysis remain almost constant until the attainment of first limit states of the case studies since the stiffness remains fairly constant until the break of infill panels. Furthermore, the loading pattern alters with the failure of infill walls, mostly in the ground floor, and concentration of displacements in that floor where the infills fail.
- The capacity curves of the bare and the infilled frames overlap in most of the cases after ultimate strain level of infill panels is reached. However, some of them do not coincide until the defined displacement of the analysis level which is due to the fact that some of the infill panels are still effective on the response of the frame, especially those existing in the upper floors which still contribute to the storey shear resistance.
- It is also apparent from some of the pushover curves such as PFN_4_2, Italian_6Str, Portuguese_60s etc., the adverse effects of infills on the response of the frame have already started after achievement of LS1.

It is important to note that the main goal of this study is to find a relationship between the first limit state of bare frames and the limit state of infilled frames corresponding to the cracking of infill panels. The reason why the latter has been compared with the former lies in the fact that the capacity for bare frames has already been defined by using various formula in displacement-based assessment methodologies and thus, this research aims to find means of simply relating the non-structural limit state for the infilled frame to the first limit state of the bare frame.

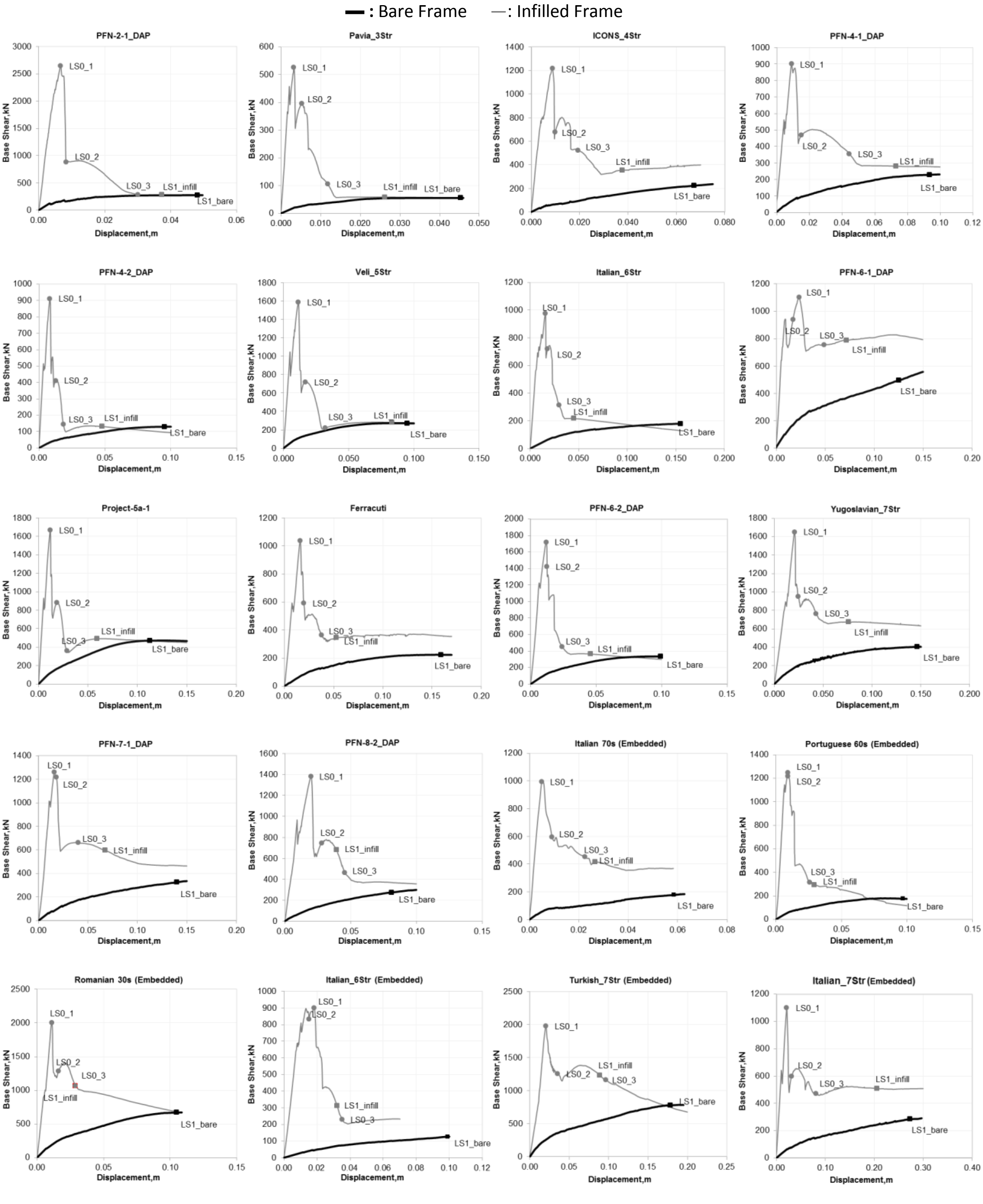



**Figure 4.1.** Capacity curves to compare the LS1limit state of the bare frame with the non-structural and structural limit states of the infilled frames

As it is observed from the capacity curves above, LS0_1 occurs at very small displacements followed by significant strength degradation, thus it is hard to relate it with the LS1 of the bare frame. Additionally, the relation of LS0_3 shows a similar trend as LS0_2 but its statistical confidence is not as satisfactory as LS0_2. Thus, LS0_2 is anticipated as the required limit state for the purpose of this study.  Besides, experimental studies conducted on the seismic response of infilled frames reveal that the lateral drift at which the masonry infill cracks is in the range of 0.07% to 0.3% (Griffith, 2008).

**Table 4.1.** Ratios between structural and non-structural limit states and statistical results

| Frames with emergent beams | | | | |
|---|---|---|---|---|
| | $\Delta_{LS1infilled}/\Delta_{LS1bare}$ | $\Delta_{LS0\_1}/\Delta_{LS1bare}$ | $\Delta_{LS0\_2}/\Delta_{LS1bare}$ | $\Delta_{LS0\_3}/\Delta_{LS1bare}$ |
| PFN_2_1 | 0.78 | 0.14 | 0.17 | 0.62 |
| Pavia_3Str | 0.58 | 0.07 | 0.12 | 0.26 |
| ICONS_4Str | 0.56 | 0.13 | 0.15 | 0.29 |
| PFN_4_1 | 0.78 | 0.10 | 0.16 | 0.47 |
| PFN_4_2 | 0.50 | 0.09 | 0.13 | 0.19 |
| Veli_5Str | 0.88 | 0.12 | 0.18 | 0.34 |
| Italian_6Str | 0.29 | 0.10 | 0.12 | 0.19 |
| PFN_6_1 | 0.57 | 0.19 | 0.14 | 0.39 |
| Project_5a_1 | 0.53 | 0.11 | 0.17 | 0.25 |
| Ferracuti | 0.33 | 0.10 | 0.12 | 0.24 |
| PFN_6_2 | 0.46 | 0.12 | 0.13 | 0.24 |
| Yugoslavian_7Str | 0.52 | 0.14 | 0.17 | 0.29 |
| PFN_7_1 | 0.48 | 0.11 | 0.13 | 0.29 |
| PFN_8_2 | 0.48 | 0.24 | 0.34 | 0.56 |
| **Mean** | **0.55** | **0.13** | **0.16** | **0.33** |
| **Standard Deviation** | **0.16** | **0.04** | **0.06** | **0.13** |
| **Coefficient of Variation** | **0.30** | **0.35** | **0.36** | **0.41** |
| **Frames with embedded beams** | | | | |
| | $\Delta_{LS1infilled}/\Delta_{LS1bare}$ | $\Delta_{LS0\_1}/\Delta_{LS1bare}$ | $\Delta_{LS0\_2}/\Delta_{LS1bare}$ | $\Delta_{LS0\_3}/\Delta_{LS1bare}$ |
| Italian_70s_3Str | 0.46 | 0.09 | 0.16 | 0.38 |
| Portuguese_60s_4Str | 0.30 | 0.09 | 0.10 | 0.27 |
| Romanian_30s_5Str | 0.27 | 0.10 | 0.15 | 0.27 |
| Italian_6Str | 0.32 | 0.18 | 0.15 | 0.35 |
| Turkish_7Str | 0.50 | 0.12 | 0.20 | 0.54 |
| Italian_7Str | 0.75 | 0.08 | 0.11 | 0.30 |
| **Mean** | **0.43** | **0.11** | **0.14** | **0.35** |
| **Standard Deviation** | **0.18** | **0.04** | **0.04** | **0.10** |
| **Coefficient of Variation** | **0.41** | **0.35** | **0.25** | **0.29** |

For the aim of comparison, the lateral drift ratios for the case studies herein are indicated in Table 4.2. The drift ratios at the suggested limit state, LS0_2 are 0.19% as maximum and 0.10 as average which are quite compatible with the experimental values. Therefore, LS0_2 is found to be the most convenient correlation, for both case studies, to be directly implemented in the loss assessment studies assuming that the deformation demands below the ones suggested through this study will not cause any substantial infill cracks and relevant financial loss.

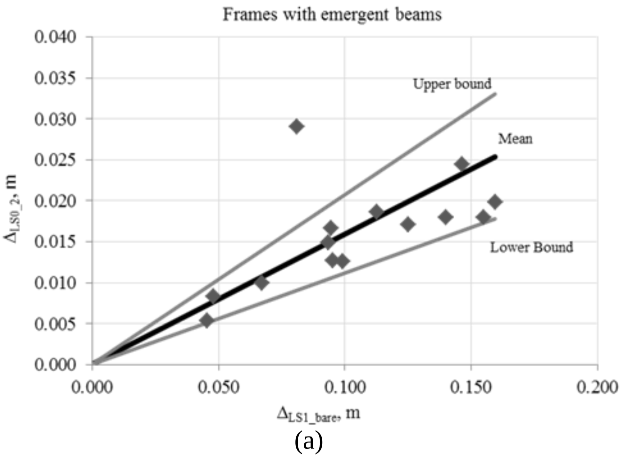
The reduction in the displacement capacity of the frames due to infill walls is also considered in order to implement this in the DBELA calculations. Accordingly, the displacement ratios at the first limit state of the infilled frame, $\Delta_{LS1\_infilled}$ to the bare frame, $\Delta_{LS1\_bare}$, are indicated in Table 4.1. Thus, the parameter, $\beta_1$ is found to have mean values of 0.55 and 0.43 for the emergent and embedded frames, respectively. The tentative average values of $\beta_2$ stated in the study of Bal [2008] are still valid since no studies have been conducted for that value in this study. However, the tentative parameter, $\beta_1$ still needs to be further calibrated with more structures and different infill arrangements.

**Table 4.2.** Drift ratios corresponding to the concerned limit states

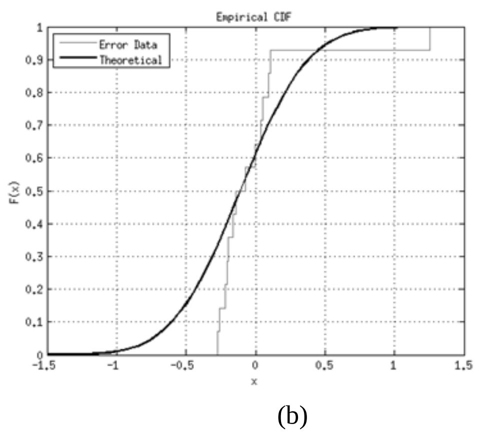
| | Drift @ LS0_1 | Drift @ LS0_2 | Drift @ LS0_3 |
|---|---|---|---|
| PFN_2_1 | 0.08% | 0.09% | 0.34% |
| Pavia_3Str | 0.05% | 0.09% | 0.20% |
| ICONS_4Str | 0.08% | 0.09% | 0.18% |
| PFN_4_1 | 0.07% | 0.12% | 0.37% |
| PFN_4_2 | 0.07% | 0.11% | 0.16% |
| Veli_5Str | 0.07% | 0.10% | 0.19% |
| Italian_6Str | 0.07% | 0.08% | 0.12% |
| PFN_6_1 | 0.14% | 0.10% | 0.30% |
| Project_5a_1 | 0.06% | 0.10% | 0.15% |
| Ferracuti | 0.09% | 0.11% | 0.20% |
| PFN_6_2 | 0.07% | 0.07% | 0.13% |
| Yugoslavian_7Str | 0.10% | 0.12% | 0.20% |
| PFN_7_1 | 0.08% | 0.09% | 0.20% |
| PFN_8_2 | 0.08% | 0.12% | 0.19% |
| Italian_70s_3Str | 0.05% | 0.10% | 0.24% |
| Portuguese_60s_4Str | 0.08% | 0.09% | 0.24% |
| Romanian_30s_5Str | 0.07% | 0.11% | 0.19% |
| Italian_6Str | 0.08% | 0.06% | 0.15% |
| Turkish_7Str | 0.11% | 0.19% | 0.53% |
| Italian_7Str | 0.08% | 0.11% | 0.29% |
| **Maximum** | **0.14%** | **0.19%** | **0.53%** |
| **Average** | **0.08%** | **0.10%** | **0.23%** |

## 4.1. Probabilistic Approach

The linear correlation obtained between LS1_bare and LS0_2 is presented by the Equation 4.1 and 4.2 below for emergent and embedded beam frames, respectively. The equations are assumed to have errors ($\varepsilon_1$, $\varepsilon_2$) from the mean indicated as upper and lower bounds. These extremes have been found to be ±30% and ±40% deviation from the mean for the frames with emergent and embedded beams, respectively (Figure 4.2 and Figure 4.3). The statistical interpretations of the errors are also conducted such that $\varepsilon_1$ has a normal distribution with $\mu$=-0.09, $\sigma$=0.14 and $\varepsilon_2$ is also normally distributed with $\mu$=0, $\sigma$=0.25.




(a)

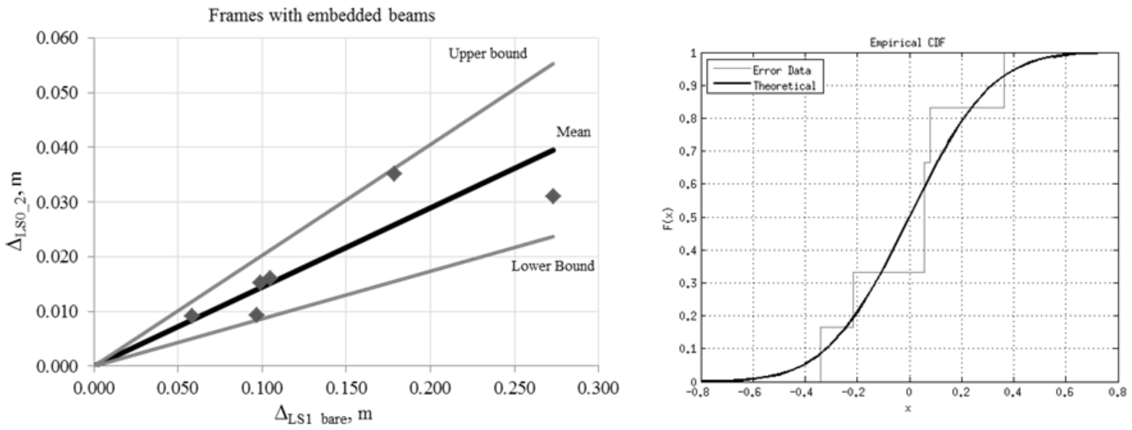



(b)

**Figure 4.2.** For frames with emergent beams, proposed linear relation between displacements of the first structural limit state LS1 and the non-structural limit state LS0 defined by attainment of $\varepsilon_m$ of the infills along with its lower and upper bounds (a) and distribution presentation of the residual, $\varepsilon_1$(b)

*For emergent beam frames:* $\Delta_{LS0\_2} = 0.16\, \Delta_{LS1\_bare} \pm \varepsilon_1$ (4.1)

*For embedded beam frames:* $\Delta_{LS0\_2} = 0.14\, \Delta_{LS1\_bare} \pm \varepsilon_2$ (4.2)




(a) (b)

**Figure 4.3.** For frames with embedded beams, proposed linear relation between displacements of the first structural limit state LS1 and the non-structural limit state LS0 defined by attainment of $\varepsilon_m$ of the infills along with its lower and upper bounds (a) and distribution presentation of the residual, $\varepsilon_1$(b)

## 5. CONCLUSIONS

Twenty case study frames selected from different regions of Turkey and Europe covering a wide range in terms of the variability of the several parameters such as construction year, design method, sectional properties, geometrical properties and material properties etc. are modelled in fibre-based nonlinear analysis software, SeismoStruct [SeismoSoft, 2010]. They are analysed by utilizing a novel analysis method, Displacement Based Adaptive Pushover proposed by Antoniou and Pinho [2004] and by Pinho [2005], which is advised to be used to overcome the unstable behaviour observed in frames with infill elements.

Upon conducting the straightforward study, an easy-to-apply formula is obtained for the distinction of the slight and no damage limit states of structures with infilled frames. The equation obtained in this study only depends on the LS1 of the bare RC frame so that the displacement capacity of the infilled frames for the needed damage state can be easily calculated without modelling, simply relying on mechanics-based displacement capacity formulae existing in the literature.

Moreover, the drift ratios corresponding to the limit state proposed for no damage level (LS0_2) have been provided and compared with the experimental test results conducted on infilled RC frames available in the literature. As a result, the obtained drift ratios in this study are in the range of the values indicated in several test results (Griffith, 2008), and it is found that the infill panels can be assumed to have negligible cracks up to the limit state where infill strain value is at maximum stress, $\varepsilon_m$, LS0_2.

## 6. FUTURE DEVELOPMENTS

The modelling of openings in infilled frames is an essential issue that should be taken into consideration. For the sake of completeness, different arrangements and dimensions of openings should be inserted into the models considered herein so that more realistic deformation capacities can be achieved in order to be implemented to the probabilistic framework of DBELA methodology.

This study may also be extended to consider 3D models, which will increase significantly the complexity of the analyses. The effects of the infill walls on such models would then present other issues for consideration such as the influence of the irregular distribution of the walls both in plan and in elevation.

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