A simplified displacement-based procedure for design and performance evaluation of low-rise masonry infilled RC frame buildings in Pakistan

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A simplified displacement-based procedure for design and performance evaluation of low-rise masonry infilled RC frame buildings in Pakistan

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ABSTRACT

RC frame structures with masonry infill walls represent a typical building topology in Pakistan and many countries worldwide. Burnt brick walls are commonly used in buildings as partitions and external walls. Practicing engineers often employ code-based procedures for the structural design of buildings and ignore the effects of infills, considering them as non-structural components. However, these infill walls contribute to the early strength, initial stiffness, and energy dissipation capacity of the frames and thus can considerably modify the global seismic performance of buildings. This contribution can be captured using nonlinear structural models. However, in the conventional code-based design procedures (based on linear elastic modeling), the infills are generally not included in the analysis model. This study presents a simplified displacement-based design (SDBD) procedure based on the concept of equivalent linearization to overcome this issue. The SDBD procedure uses the framework of the direct displacement-based design (DDBD) method to determine the equivalent linear characteristics of the nonlinear structure. The fundamental idea is that the seismic demands of a nonlinear system can be determined using an equivalent linear system of elongated period and higher damping. The proposed procedure is developed using the three case study buildings (3-, 4-, and 5-story high), and its accuracy is investigated for a 5-story high building under input ground motions. In this evaluation, the inelastic demands obtained from the nonlinear analysis procedure are employed as a benchmark. The proposed SDBD procedure is found to provide accurate estimations of local and global responses for the case study building.

KEYWORDS

Equivalent linearization, masonry infill frames, nonlinear analysis, displacement-based design procedure

1. INTRODUCTION

Pakistan lies in an active Himalayan orogenic belt, formed by the slow interaction among the Arabian, Indian, and Eurasian tectonic plates $30 - 40$ million years ago (Aitchison et al., 2007). Moderate to severe earthquakes are frequent in this region, mainly due to tectonic deformations. The earthquakes will continue as long as the interaction among the tectonic plates continues. Pakistan is thus affected by several earthquakes of various magnitudes, making it one of the top fifty active countries by seismic hazard around the world (Tsapanos & Burton, 1991). The most common structural configuration in Pakistan and many developing countries is masonry infilled reinforced concrete (RC) frame buildings, with infill walls encased between structural frame elements (Lodi et al., 2013; Polese et al., 2020). It is due to their ease of construction and requiring lesser technical skills. Two different approaches are in use to construct the infill RC frame buildings. One approach is to insert the plastic material between the RC frame and infill wall so that no connection exists between these two elements. Furthermore, they can deform independently under the design hazard level, besides at the base of the infill. This approach requires the provision of some reinforcement in infill walls as well as out-of-plane support (Paulay & Priestley, 1992). In the alternate technique, masonry infill walls are bonded to the frame elements, and their (RC frame and infill) structural interaction is considered in the design stage. Generally, the bonded infill walls are more vulnerable to damage because they are unreinforced and may fail at comparatively low drift values. In Pakistan, where infill walls are placed in frame elements with a strong bond, the failure of infill walls is considered non-structural, and the damage is repaired after the earthquake. There are no guidelines or recommendations for designing the masonry-infilled RC frame structures in the Building Code of Pakistan (BCP, 2007). Therefore, during the structural design of buildings, masonry infill walls are disregarded, assuming that infill walls have no structural significance.

In Pakistan, the design and evaluation of infill walls in infill frames solely focus on their estimated gravity loads, neglecting all other critical aspects such as relative stiffness and strength of infill walls & RC frames. Ignoring the contribution of infill walls to structural performance will result in significant strength prediction inaccuracy because these infill walls can completely alter the damage distribution throughout the building (Dolsek & Fajfar, 2008; Pantò et al., 2017). The presence of masonry infill walls affects the global seismic performance of the structure by imparting initial lateral translational stiffness, initial lateral bearing strength, and enhanced energy dissipation capacity at low drift levels (Rodrigues et al., 2010; Stafford Smith & Carter, 1969). It is conceivable as long as

seismic demands do not exceed the deformation capacities of infill walls. During the recent earthquakes, the damage observed revealed that newly built infill frames designed with seismic provisions are as vulnerable as older built infill frames. It is due to the adverse effects of infill walls and RC frames interaction (Butenweg et al., 2019). The fundamental natural period of the structure is considerably reduced due to the presence of infill panels because infill panels tend to provide high initial stiffness to the building. In the response spectrum analysis (RSA) method, the reduced natural period will cause the change in demand forces on the building (Meharbi & Shing, 2003). In some cases, infill frames exhibited up to three times more stiffness and base shear relative to bare frames, depending on the stiffness and distribution of infill panels in the building (Sigmund & Penava, 2014). When the distribution of infill walls is improper in the structural plan, it may be the most dangerous situation. The irregular or non-uniform distribution of infill panels might cause undesirable consequences like additional torsional effects and brittle shear failure of columns (Dolsek & Fajfar, 2008; Paulay & Priestley, 1992). The presence of infill walls has a considerable impact on building behavior at the local level as well. The interaction between infill and surrounding frame may change the distribution of internal forces in structural elements leading to brittle collapse mechanisms (e.g. short column effects) and even collapse of self-stable frame structures (Layadi et al., 2020). These infill walls are ineffective in sustaining lateral displacements imposed by strong earthquake shakings due to their non-ductile behavior.

Due to the behavior of infill walls being strictly brittle in nature, it is not wise to model the infill walls with the linear elastic modeling technique. A nonlinear force-deformation behavior that is sufficiently sophisticated is required to capture the failure modes of masonry infill walls to account for its effects on the structure. Compared to linear elastic modeling, nonlinear modeling of buildings demands a substantially superior level of expertise and a deep understanding of different complex interactions. It additionally requires a great deal of effort and computational resources. The selection of representative ground motions to perform the detailed nonlinear time history analysis (NLTHA) procedure and post-processing the results can take considerable time. In addition, a typical design firm may lack the relevant skills and resources required to go through this process for every project. In a nutshell, currently, there is no practical method available to consider the effects of these infill walls in the design and assessment of frame structures. Therefore, a feasible approach considering the effects of masonry infill walls on the response of frame buildings is required.

Considering this gap, this study proposes a practical analysis scheme incorporating the effects of infill walls for the performance evaluation of existing buildings and the design of new buildings. It is based on the direct displacement-based design (DDBD) method of Priestley for RC infill frames (Priestley et al., 2008). The primary idea of the DDBD method is that structures must be designed to achieve a certain level of performance, in terms of drifts or strain limits, under a certain level of hazard. This philosophy would result in "uniform risk" structures, which is consistent with the uniform risk spectra of code-based design methods. The DDBD method is based on the substitute structure approach developed by Gulkan & Sozen (1974). The substitute structure approach narrates that properly tuned linear elastic SDOF systems (with the elongated period and additional damping) can approximately represent the response of nonlinear SDOF systems. As a result, nonlinear seismic demands of SDOF systems can be determined with the desired accuracy following this approach. The DDBD method has previously been used to determine the nonlinear seismic demands of concrete masonry infill frames. However, it does not account for the effects caused by masonry infill panels in RC frames when subjected to other than low drift levels. In this study, a convenient iterative scheme for effectively determining the seismic demands of existing low-rise infill RC frame buildings is presented. It retains the simplicity of the linear elastic modeling technique for practicing engineers while providing satisfactory results compared to the NLTHA procedure.

2. THEORETICAL FORMULATION

For the design and assessment of structures, force-based design procedures have been broadly used currently and in the past. The major shortcoming of the force-based design procedure is that it uses initial stiffness K_i to distribute the forces in structural elements (stiffer elements attract more forces) and employs an initial level of damping ξ_{el} in the energy dissipation process. It does not provide the guidelines to model the actual nonlinear behavior of elements and implicitly accounts for the inelastic behavior through the use of R, C_d, and Ω. Here, R is the response modification factor, C_d is the displacement amplification factor, and Ω is the over-strength factor to be applied to the structural members. On the other hand, the behavior of masonry infill walls is strictly nonlinear, and this behavior can't be captured by linear elastic modeling of force-based design methods.

Therefore, an established method named the DDBD procedure is considered as a theoretical foundation for proposing a practical approach to design and evaluate the infill RC frames. The DDBD method uses the secant stiffness K_e and equivalent damping ξ_{eq} to estimate the actual nonlinear seismic demands on the structure. The

DDBD method is based on the equivalent linearization (EL) technique, which is also the basis of the proposed procedure. Consequently, it is essential to concisely review the fundamental concepts and basic assumptions of the DDBD method.

2.1. The Direct Displacement Based Design Method (Priestley et al., 2008)

The direct displacement-based design (DDBD) method has undergone extensive development to mitigate the deficiencies in force-based design and analysis procedures. Rather than designing the actual structure by its initial elastic characteristics, the DDBD method uses a single degree of freedom (SDOF) system characterized by secant stiffness K_e. The secant stiffness K_e is significantly lower than the initial stiffness K_i, representing the performance at the peak response level. Equivalent viscous damping ξ_{eq}, the combination of elastic damping ξ_{el} and hysteretic inelastic response damping ξ_{hys}, is employed to be compatible with the secant stiffness K_e idea. Considering secant stiffness K_e instead of initial stiffness K_i results in higher peak displacements for all of the hysteresis rules studied. The DDBD is based on the substitute structure approach that idealized the building as an equivalent linear SDOF system (Gulkan & Sozen, 1974). Subsequently, this approach was improved by considering the building as a multi-degree of freedom (MDOF) system (Shibata & Sozen, 1976).

To convert an original nonlinear SDOF system into an equivalent linear system, the equivalent linear system must be characterized by the equivalent stiffness K_e at the maximum displacement response. The underlying idea is that a nonlinear system and its equivalent linear system should have approximately similar energy dissipation characteristics. Fig. 1 illustrates the basic concept of converting the nonlinear SDOF system into an equivalent linear SDOF system. The symbols used in Fig. 1 are explained in Section 5 later, where equivalent linear properties T_{eq} & ξ_{eq} are determined.

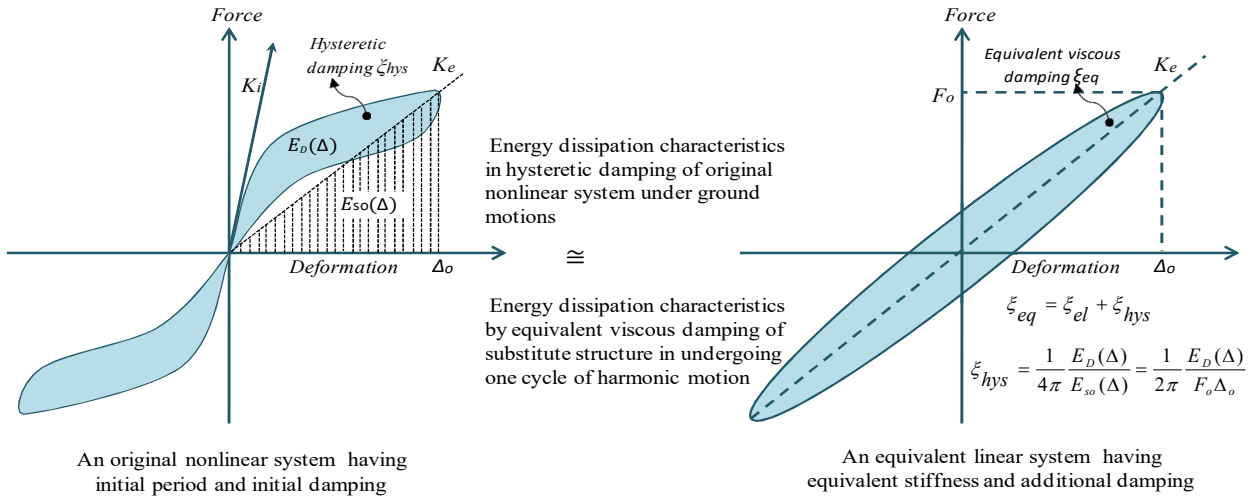


Fig. 1 Conversion of nonlinear SDOF system into an equivalent linear system

The procedure of the DDBD method is shown in Fig. 2, with an emphasis on SDOF representation of frame structure, while the basic concepts apply to all structural types. The lateral force-displacement response of frame building represented by the SDOF system is displayed as a bilinear envelope where initial lateral stiffness K_i changes to post-yield stiffness rK_i.

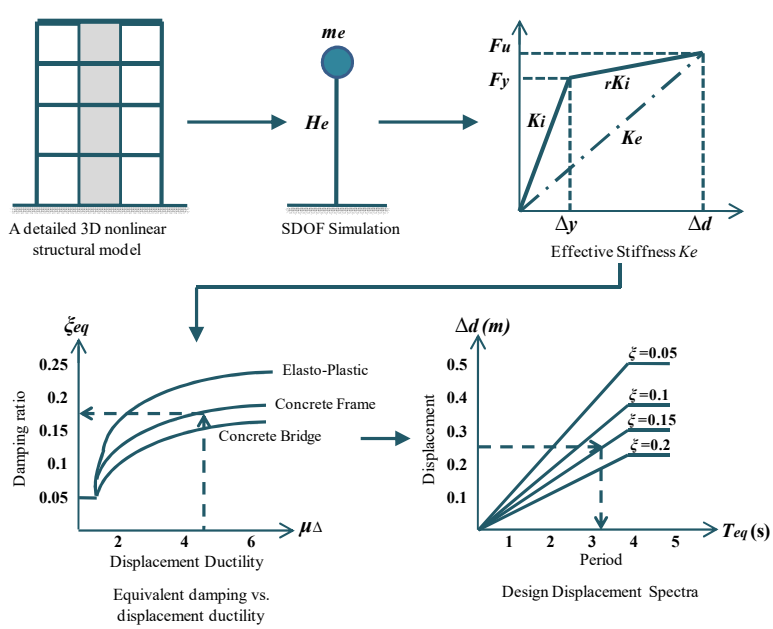


Fig. 2 Fundamentals of the DDBD method (Redrawn from Priestley, 2008)

The DDBD method requires the design of structures for the severe level of ground shaking. The lateral displacement demands Δ_d on the building can be estimated using the assumed inter-story drift (IDR) profile and employing the

critical inter-story drift ratio (IDR). The yielding lateral displacement Δ_y of the structure can be obtained using the slenderness ratio of the beams. After the determination of displacement ductility μ_Δ of the building using Δ_y and Δ_d, the equivalent viscous damping ξ_{eq} can be estimated using μ_Δ and structural type, as shown in Fig. 2. A series of displacement spectra can be developed for various levels of equivalent damping ξ_{eq} to determine the effective period T_{eq} corresponding to the effective height H_e. The effective stiffness K_e of the analogous SDOF system at the peak response level can be obtained by employing the following equation:

$$\boldsymbol{K_e = \frac{4\pi^2 m_e}{T_{eq}{}^2}} \tag{1}$$

Here m_e represents the effective mass of the building, which is participating in the first mode of vibration. The design lateral force or design base shear can be determined using K_e and Δ_d as follows:

$$F = V_{base} = K_e\, \Delta_d \tag{2}$$

The design base shear estimated from the above equation is then distributed to different floors based on the product of mass and displacement as follows:

$$F_i = V_{base} \frac{(m_i \Delta_i)}{\sum_{i=1}^{n} (m_i \Delta_i)} \tag{3}$$

The structure is then analyzed using the force vector characterized by equation (3) to estimate the requisite flexural demands at designated plastic hinges. The DDBD method estimates the loading demands at specified plastic hinge locations to meet the design objectives in terms of target displacements. Capacity design procedures are required to ensure that plastic hinges form only at those locations where appropriate detailing for ductility demand is provided; brittle modes of deformation (shear failure of frame elements) don't occur. The complexity exists in determining the equivalent linear properties of substitute structure, estimating the maximum design displacement, and developing the design displacement spectra. Several studies (Malekpour et al., 2011; Sharma et al., 2020) have applied the DDBD method to case study frame buildings and assessed the accuracy by comparing the responses obtained from the DDBD method to the detailed NLTHA procedure. These studies concluded that the DDBD method, with the known hysteretic behavior of the structure under consideration, can effectively estimate the floor displacements, inter-story

drift ratios, story shears, and story overturning moments. Detailed information on the DDBD method and its validation can be found in Priestley et al. (2008).

A two-stage design scheme is required when masonry infill walls are placed in frame structures. In the serviceability level earthquake (SLE), infill failure does not occur, so the structure should be modeled linear elastically to get realistic stiffness estimates. But during the maximum considerable earthquake (MCE), only the bare frame is responsible for withstanding ground shakings. The DDBD method assumes that masonry infill walls have no structural significance during the severe level of the earthquake. The rational approach is to design the infill frames without considering the infill walls in the MCE level of earthquake. The bare frame must be subsequently detailed to provide the required capacity during severe ground shakings.

2.2. The Proposed Simplified Displacement Based Design (SDBD) Procedure

The underlying assumption of the SDBD procedure is that an equivalent linear elastic system can mimic the response of a nonlinear inelastic system, provided that both systems are at the maximum displacement response level. However, unlike the DDBD method, where design displacement Δ_d is determined using the critical IDR and effective height H_e, the displacement demand Δ_d on the building in the SDBD procedure is determined using an iterative scheme, explained later in this section.

The DDBD method developed the general relations as a function of ductility ratio ($\mu_\Delta = \Delta_d / \Delta_y$, where Δ_y is the yielding displacement) for different structural systems to estimate the equivalent linear characteristics T_{eq}& ξ_{eq}. However, the SDBD procedure proposes the generalized relationships to determine T_{eq} & ξ_{eq} as a function of roof drift ratio RD instead of ductility ratio μ_Δ. Recent studies (Mehmood et al., 2017; Munir & Warnitchai, 2011) have indicated that significant nonlinearity is induced during the cyclic response of RC structures due to tensile cracking of shear walls before the yielding of steel reinforcement. Tensile cracking of RC shear walls resulted in considerable stiffness softening at relatively lower displacements than the yielding displacement Δ_y. In these situations, where there's no well-defined yield point in buildings, the ductility ratio μ_Δ may not be an appropriate parameter to represent the nonlinear structural state. The development of generalized relations with a suitable deformation parameter is of prime importance to ensure the practical implication of the SDBD procedure in design offices.

Therefore, the roof drift ratio RD is used, which is a more appropriate and meaningful index to represent the nonlinear state of a structure.

The fundamentals of the SDBD procedure are illustrated in Fig. 3 for the infill RC frame structures, while the primary concept implements on any structural configuration. The RD_{el} is the roof drift ratio of the building when it is in the linear elastic range, and T_{el} & ξ_{el} are corresponding elastic natural period and damping characteristics of the building. The initial natural period T_o of the building can be determined from the modal analysis of the linear elastic model. The period T_o is used as an input to the acceleration response spectrum developed for ξ_{el} damping to estimate the seismic demands.

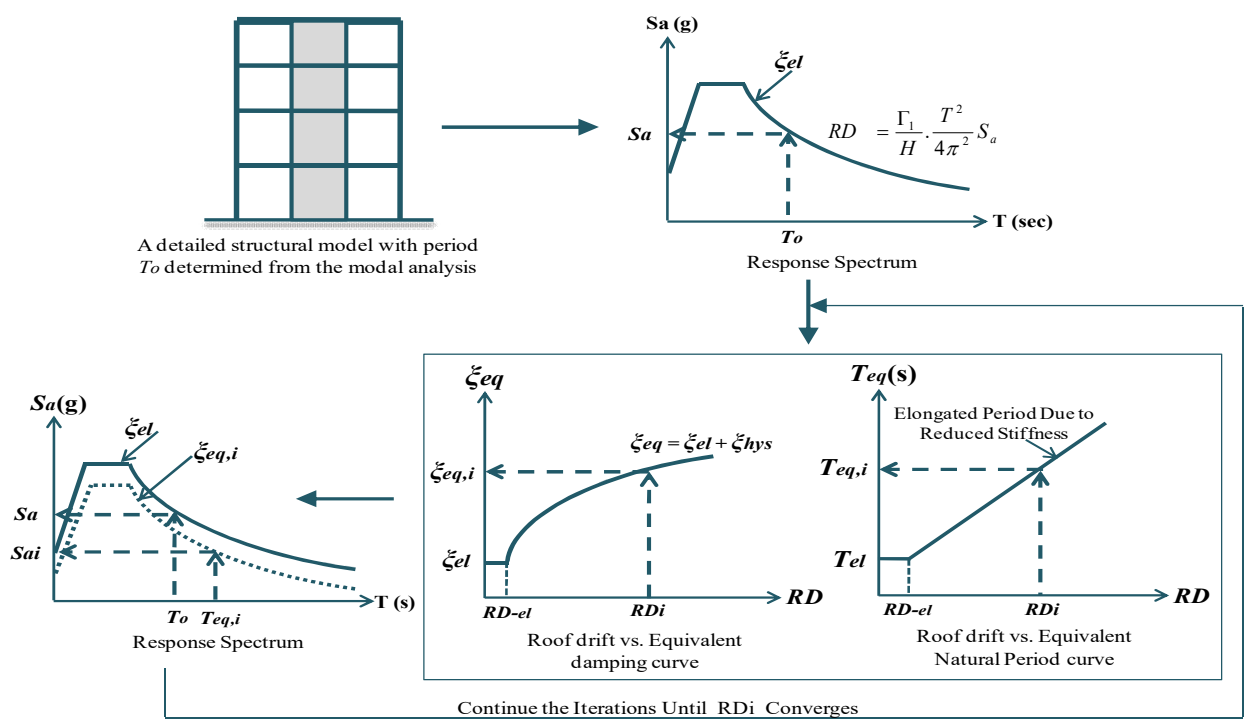


Fig. 3 Basic concepts of the SDBD procedure

The acceleration response spectrum can be converted into the displacement response spectrum using the following equation:

$$S_d = \frac{1}{\omega^2} S_a = \frac{T^2 S_a}{4\pi^2} \tag{4}$$

Here S_d represents the spectral displacement, S_a represents the spectral acceleration, and T shows the natural period of the building.

The elastic roof drift RD_{el} can be determined from spectral parameters by the use of this equation as follows:

$$\boldsymbol{RD_{el} = \frac{\Gamma_1}{H} S_d = \frac{\Gamma_1 . T^2 . S_a}{4\pi^2 . H}} \tag{5}$$

Here Γ_1 represents the modal participation factor of first vibration mode and H is the total height of the building.

The trial roof drift, usually RD_{el}, is used to get a trial pair of $T_{eq,i}$ & $\xi_{eq,i}$ from the corresponding relationships. The response spectrum is reduced for the pre-determined $\xi_{eq,i}$, and the roof drift RD_i corresponding to $T_{eq,i}$ is obtained. The roof drift ratio RD_i is updated by again estimating the spectral displacement S_d associated with the trial $T_{eq,i}$ & $\xi_{eq,i}$. This process is repeated until the initial value of RD_i for the iteration converges to the resulting RD_i yielding the final equivalent linear characteristics of the building. The convergence has occurred in a maximum of three trials for the case study buildings. Equation (1) of Section 2.1 is employed to obtain the secant stiffness K_e from the equivalent natural period T_{eq}. The final roof drift ratio RD is converted into design displacement Δ_d using the total height of the building H as follows:

$$\Delta_d = RD \times H \tag{6}$$

The design lateral force on the building is determined using equation (2) of Section 2.1. The determined design force is distributed to different floor levels as per the distribution method based on the first mode of vibrations as follows:

$$F_x = V_{base} \frac{(w_x h_x)}{\sum_{i=1}^{n} (w_i h_i)} \tag{7}$$

Here, F_x is the lateral force at level x of the building, w_x & w_i are effective weights of the building at level x or i, and h_x & h_i are heights of the building at level x or i. The structure is then analyzed using the force vector characterized by equation (7) to evaluate its performance.

Once the efficiency of the proposed SDBD procedure in determining nonlinear seismic demands is established, the representative curves for various structural systems can be constructed in future studies. The generalized relationships for different structural systems can be developed by employing an existing EL approach combined

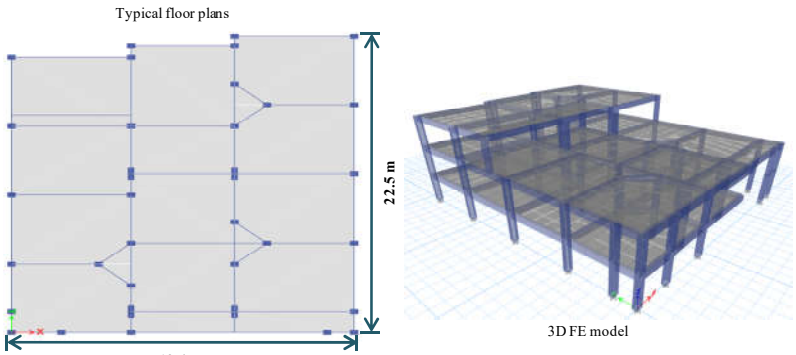
with appropriate hysteretic behavior or tuning the responses obtained from dynamic analysis. In summary, the proposed SDBD procedure can be carried out directly without developing the nonlinear structural model and without determining the nonlinear lateral cyclic response of buildings.

3. DESCRIPTION AND MODELLING OF CASE STUDY BUILDINGS

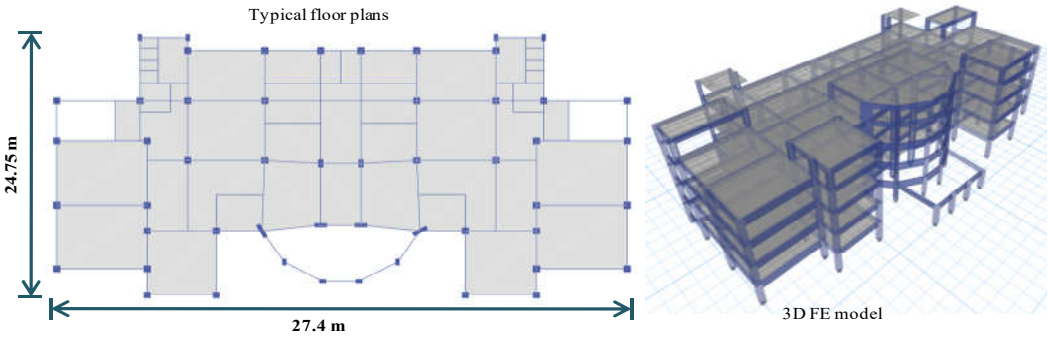
The SDBD procedure is proposed to design new buildings and evaluate the seismic performance of existing buildings. As a design procedure, it aims to provide accurate estimations of actual nonlinear seismic demands of new buildings to determine the requisite lateral capacity to withstand earthquake loadings. After providing the required strength in structural elements at critical defined locations as per demands determined from the SDBD procedure, the seismic performance evaluation of the building can be carried out using its equivalent linear characteristics.

In this study, three existing low-rise case study buildings with masonry infill walls are selected to propose the SDBD procedure. The case study buildings (3-, 4- and 5-story high, designated as B1, B2, and B3, respectively) are located in Islamabad, the capital city of Pakistan, and are already designed against gravity and seismic loads. They can represent typical existing RC frame buildings in many developing countries across the globe. The thickness of interior and exterior masonry infill walls is 9 inches in case study buildings. A 5-story high infill RC frame building (designated as B4), also located in Islamabad, is used to evaluate the performance of the proposed SDBD procedure in nonlinear seismic demands estimation. The detailed NLTHA procedure is performed using a compatible set of ground motion histories. The nonlinear seismic demands determined from the NLTHA procedure are employed as a benchmark to examine the validity of the proposed SDBD procedure. The seismic demands obtained from the response spectrum analysis (RSA) method are also included in the comparison.

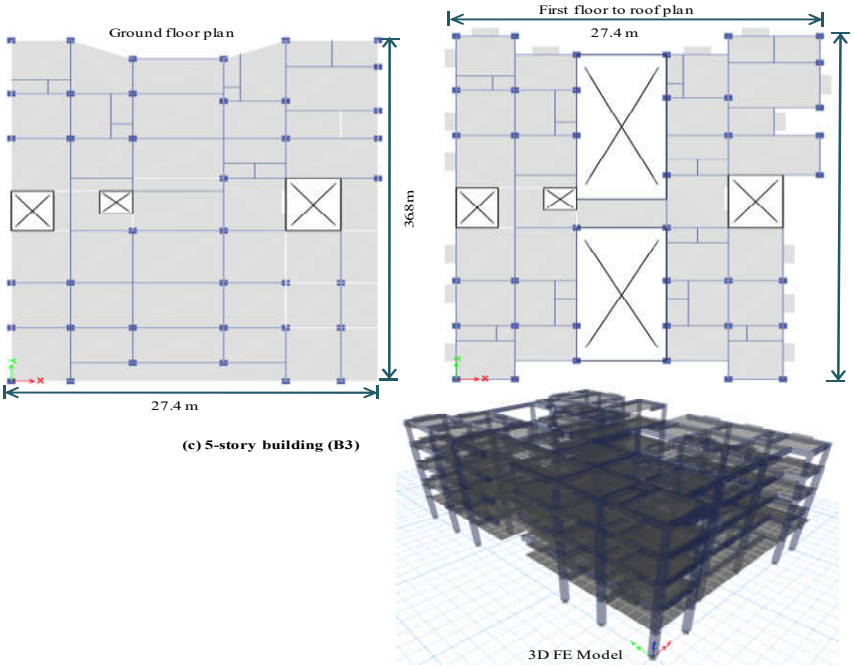
The typical floor plans with 3D finite element (FE) models of all B1, B2, B3, and B4 case study buildings are presented in Fig. 4. In these case study buildings, the gravity loads are resisted by RC beam-column frames and RC slabs, whereas the lateral loads are primarily resisted by RC columns. The case study buildings have isolated foundations at the base of columns. The masonry infill panels are provided on interior and exterior frames of buildings. Because case study buildings are practical and realistic examples of building stock, these selected buildings are considered appropriate to propose and examine the efficiency offered by the SDBD procedure.



(a) 3-story building (B1)



(b) 4-story building (B2)



(c) 5-story building (B3)

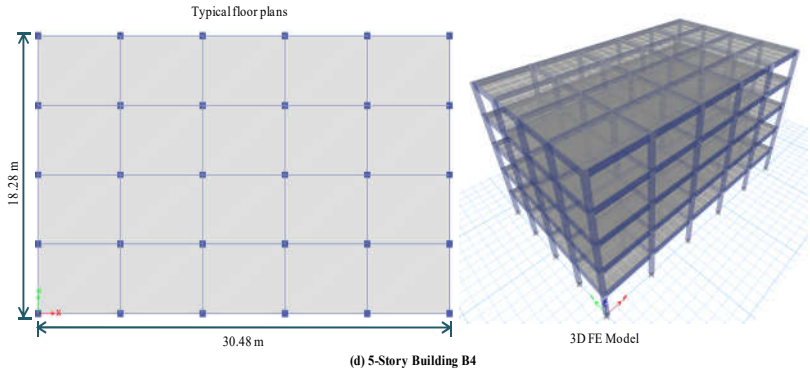


Fig. 4 Plans and 3D views of case study buildings

Major architectural and structural characteristics of selected buildings are presented in Table 1.

Linear elastic structural models of the case study buildings are developed using the ETABS (Computers & Structures, 2018) software to carry out the SDBD procedure and the standard RSA method. All the columns and beams are modeled as elastic frame elements, while all the slabs are modeled as elastic shell elements. The masonry infill walls are modeled linear elastically using the equivalent concrete diagonal struts. The struts are capable of developing the axial compression behavior only. The properties of masonry infill walls used to model the diagonal struts in the context of Pakistan (ASHRAF, 2010; Javed, 2009; Khan, 2018; Shahzada et al., 2012) are displayed in Table 2. The isolated foundation at the base of the ground floor columns is considered fixed support. For the SDBD procedure, code-prescribed stiffness modifiers to account for cracking before yielding are assigned to structural frame elements to effectively estimate the natural periods (T) and subsequent responses under seismic excitations.

Table 1 Salient features of case study buildings

Building			B1	B2	B3	B4
Height (m)			9.9	15.62	17.4	18.28
No. of stories			3	4	5	5
Typical story height (m)			3.3	3.28	3.35	3.65
Total footprint area (m × m)			19.35 × 22.52	51.81 × 24.75	27.38 × 36.8	30.48 × 18.28
Natural periods (sec)	X-direction	T1	0.2287	0.3702	0.4482	1.035
		T2	0.0823	0.1226	0.1388	0.3379
		T3	0.0598	0.0778	0.0718	0.1957
	Y-direction	T1	0.1902	0.3494	0.3987	0.7705
		T2	0.07602	0.1165	0.1317	0.2552
		T3	0.0577	0.0717	0.0759	0.1477
RC column area/ total foot- print area (%)			1.417	2.08	2.821	1.13
Infill thickness (m)			0.2286	0.2286	0.2286	0.2286
Typical column size (mm × mm)			457 × 355	609 × 609	609 × 914	457 × 457

Table 2 The properties used to model the masonry infill walls

Symbol	Properties	Value
f_m	Masonry compressive strength (MPa)	4.4
f_{tu}	Masonry tensile strength (MPa)	0.15
E_m	Elastic modulus of masonry (MPa)	1310
f_{vi}	Masonry shear strength (MPa)	0.27

The detailed nonlinear finite element (FE) models of the case study buildings are developed using the PERFORM 3D (Computers & Structures, 2018) software to carry out the cyclic pushover analysis and the detailed NLTHA procedure. All the RC columns are modeled by distributed plasticity approach to save computational effort and time, using the nonlinear fiber elements at their two ends. Fibers are created in both cross-sectional dimensions of

columns to account for biaxial bending, as illustrated in Fig. 5. This fiber model accounts for axial-flexure interaction ($P - M_x - M_y$) in both axes. The shear and torsion behavior is assumed to be elastic and uncoupled from the axial-flexure interaction. The shear force demands to capacity (D/C) ratios are checked after the analysis to ensure no pre-mature shear failure occurs in column members. The plasticity is distributed at both ends for a length of D, where D is the shorter dimension of the column cross-section. The unassigned portion of column length is treated as a linear elastic element. The concrete fibers of column elements are modeled by employing Mander's stress-strain model of unconfined concrete (Mander et al., 1988) idealized as a tri-linear behavior in PERFORM 3D software, as shown in Fig. 5. The bilinear stress-strain model with the strain hardening effect is used to model the steel fibers, as displayed in Fig. 5.

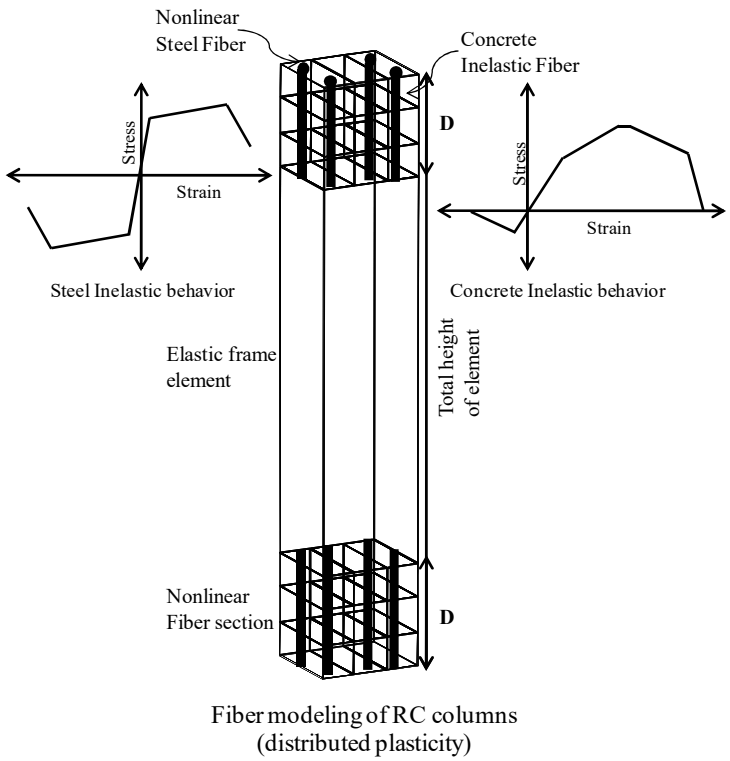


Fig. 5 Fiber modeling of RC columns in case study buildings

All the RC beams are modeled using the moment rotation ($M - \theta$) plastic hinge modeling approach following the ASCE 41 (2017), where inelastic behavior concentrated at ends is defined by zero-length hypothetical elements (plastic hinges), as shown in Fig. 6. Several types of plastic hinges are available in PERFORM 3D software to specify the inelastic force-deformation behavior at designated locations of the member for any degree of freedom. Here, the M3 degree of freedom (DOF) is considered critical in RC beam elements, and ($M - \theta$) rigid plastic hinge is used to assign nonlinear force-deformation behavior to this DOF only. The remaining portion of beam elements is

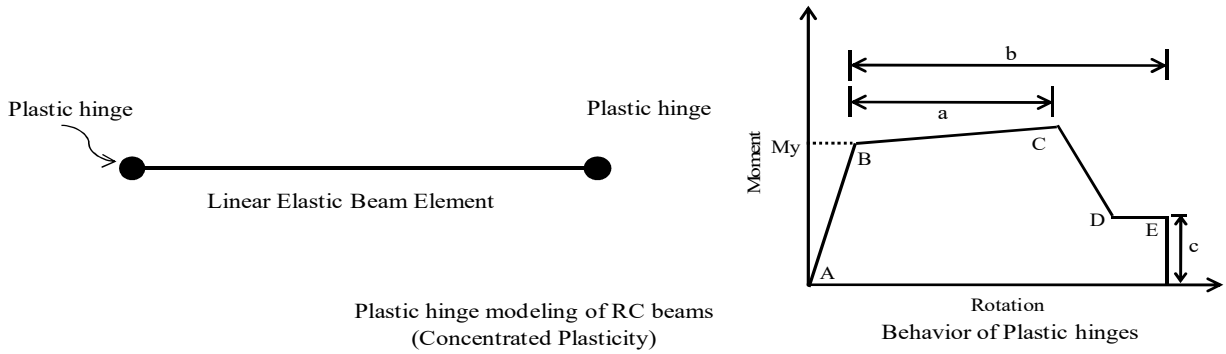


Fig. 6 ASCE 41 hinge modeling approach for RC beams (a, b, c are modeling parameters)

considered linear. Fig. 6 illustrates the key features of a flexure hinge model at the member level for RC beam elements with inelasticity concentrated at ends. The backbone curve and cyclic behavior of the material under consideration are required to define the plastic hinges in PERFORM 3D software. The parameters for cyclic degradation of concrete structural elements are taken from the Lowes et al. (2018) study.

In the initial level of seismic response, an infill wall that is built-in contact with the frame elements on all sides acts as a diagonal strut. Fig. 7 illustrates that masonry infill panels are modeled with two equivalent concrete diagonal struts with brittle compression behavior only. This diagonal strut model provides the overall satisfactory stiffness

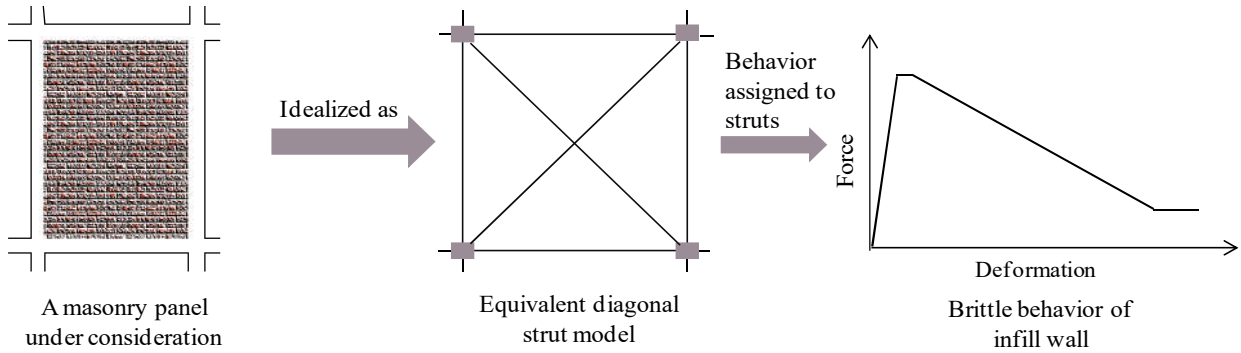


Fig. 7 Modeling of infill walls using equivalent concrete diagonal struts

estimation of infill frames and the axial forces induced in frame elements due to excessive in-plane stiffness of infill panels under lateral loadings (Crisafulli, 1997). Under earthquakes, significant compressive stresses develop at the ends of the masonry strut, and the other two corners experience tensile stresses, which lead to the separation of the infill wall and the frame. The infill panels crack in various patterns as the lateral loading increases. The stiffness, axial strength, and nonlinear deformation capacity of all masonry infill walls modeled using diagonal struts are obtained from the material and geometrical properties of these walls as per the FEMA (2000) guidelines. The expected strength of material properties is used in all the nonlinear models of buildings considering that the actual capacity of materials usually exceeds the nominal capacity assigned by the designer. Therefore, the yielding strength of steel is multiplied by "1.17", and specified concrete compressive strength is multiplied by "1.3" as per LATBSDC (2017) guidelines.

4. COMPUTATION OF EQUIVALENT LINEAR PROPERTIES

As previously stated, any nonlinear system governed by the modal hysteretic response can be transformed into an equivalent linear system. Even though the case study buildings have different total heights, footprint areas, and structural arrangements, the sequence of damage accumulation and global hysteretic responses of case study buildings are highly comparable. It implies that comprehensive relationships can be developed to determine the equivalent linear characteristics as a function of suitable deformation measure (e.g. roof drift ratio RD) for infill frames. For this purpose, nonlinear models of the case study buildings are subjected to cyclic modal pushover analysis (MPA) for the 1st mode in the X-direction. In cyclic MPA, each incremental load cycle was applied to case study buildings with the stiffness characteristic carried over from the end of the previous load cycle. The resulting cyclic pushover curves (V_{base}/W vs. RD) are used to develop the effective period T_{eq}/T_o versus roof drift RD relations, as shown in Fig. 9 (where T_o is the initial period of the building). Similarly, hysteretic damping ξ_{hys} versus roof drift RD relationships are also developed from the cyclic pushover curves of buildings to complete the set of equivalent linear characteristics (T_{eq} & ξ_{eq}).

Fig. 8 displays the base shear coefficient (V_{base}/W) versus roof drift ratio RD curves of monotonic and cyclic pushovers for the first mode in the X-direction of all case study buildings. The monotonic pushover curves (V_{base}/W vs. RD) of the buildings are transformed into force-displacement curves. The slope of these force-displacement curves is secant stiffness K_e, from which the effective period T_{eq} is determined using equation (1). It resulted in

T_{eq}/T_o vs. RD curves for all three case study buildings, as shown in Fig. 9. Fig. 9 indicates that case study buildings exhibit a similar trend of equivalent linear period T_{eq} normalized to the initial period T_o. It further implies that a generalized equation to determine the equivalent period T_{eq} can be developed for masonry infill RC frames. The proposed equation is as follows:

$$T_{eq} = T_o[\frac{15(RD_{Total} - RD') + 0.14}{(RD_{Total} - RD') + 0.127}] \tag{8}$$

Here, RD_{Total} is the total roof drift ratio of the building where the equivalent linear period T_{eq} is required to be determined. RD' is the roof drift of the building structure, after which nonlinearity starts developing in the building. This equation is established using the average curve of T_{eq}/T_o of three case study buildings, and it is valid to calculate the equivalent period T_{eq} for infill frames till the following condition holds:

$$RD_{Total} - RD' \geq 0 \tag{9}$$

The determination of the equivalent period T_{eq} from the secant stiffness K_e at the given maximum nonlinear displacement leaves the additional damping estimation problem. The total equivalent viscous damping ξ_{eq} of a structure is the addition of initial inherent elastic damping ξ_{el} and hysteretic damping ξ_{hys} of inelastic response as given:

$$\xi_{eq} = \xi_{el} + \xi_{hys} \tag{10}$$

The equal energy principle initially proposed by Jacobsen (1930) can be implemented to determine the amount of hysteretic damping ξ_{hys} during the inelastic response. Jacobsen suggested that an equivalent linear system having additional damping can simulate the steady-state response of the nonlinear system. Jacobsen's original concept was that equivalent linear and nonlinear systems should have the same initial stiffness K_i. Moreover, equivalent linear and nonlinear systems must experience constant amplitude of sinusoidal excitation, both systems should be at resonance, and both systems should dissipate the same amount of energy in each cycle for accurate mimicking of the response. These premises are rarely satisfied as actual ground motions have diverse frequency content and are non-harmonic in nature. The assumption that both systems are at resonance is inconsistent with the determination of hysteretic damping ξ_{hys} from cyclic modal pushover analysis. But studies demonstrated that it is a reasonable approximation to believe that hysteretic damping ξ_{hys} is independent of the loading frequency (Chopra, 2007). The

use of equivalent damping predicted the displacements under seismic loadings in good agreement with the time history results for thin hysteretic response systems like the Takeda model for concrete frame structures (Dwairi et al., 2007).

This study has adopted Jacobsen's linearization approach with secant stiffness K_e rather than initial stiffness K_i to determine the hysteretic damping ξ_{hys}. The determination of damping using K_e was initially proposed by Rosenblueth & Herrera (1964), which was then extensively adopted in numerous other studies. Therefore, this study is compatible with the underlying concepts of characterizing the structure by viscous damping and stiffness at the peak response level. The following expression is used for the estimation of hysteretic damping ξ_{hys}:

$$\xi_{hys} = \frac{E_D(\Delta)}{4\pi E_{so}(\Delta)} = \frac{E_D(\Delta)}{2\pi F_o \Delta_o} \tag{11}$$

Here E_D is energy dissipated within one complete cycle of stabilized force-displacement response (area of that cycle). F_o & Δ_o are the maximum inelastic forces and peak displacements, respectively, obtained in each hysteretic cycle, as shown in Fig. 1. $E_{so}(\Delta)$ represents the strain energy of an equivalent linear system, which is determined from the area of the triangle of the force-deformation curve, as shown in Fig. 1. It is clear that the equivalent damping ξ_{eq} or strain energy $E_{so}(\Delta)$ is proportional to the secant stiffness K_e at peak response. A higher assumed equivalent stiffness K_e leads to lower equivalent damping ξ_{eq}, whereas a lower choice of equivalent stiffness K_e provides a higher level of equivalent damping ξ_{eq}, ensuring equal energy dissipation. The nonlinear and equivalent linear systems are likely to have the same damping force; any EL approach employing the secant stiffness K_e should modify the initial damping ξ_{el} to make the results compatible with the results of the NLTHA procedure. However, this study does not consider this modification, and equivalent damping ξ_{eq} is computed by adding the initial damping ξ_{el} to the hysteretic damping ξ_{hys}. This adopted methodology may not be the most accurate in determining the equivalent linear properties of a nonlinear system. However, it keeps the theoretical base and conceptual clarity that makes it suitable for evaluating the proposed SDBD procedure of infill frame buildings.

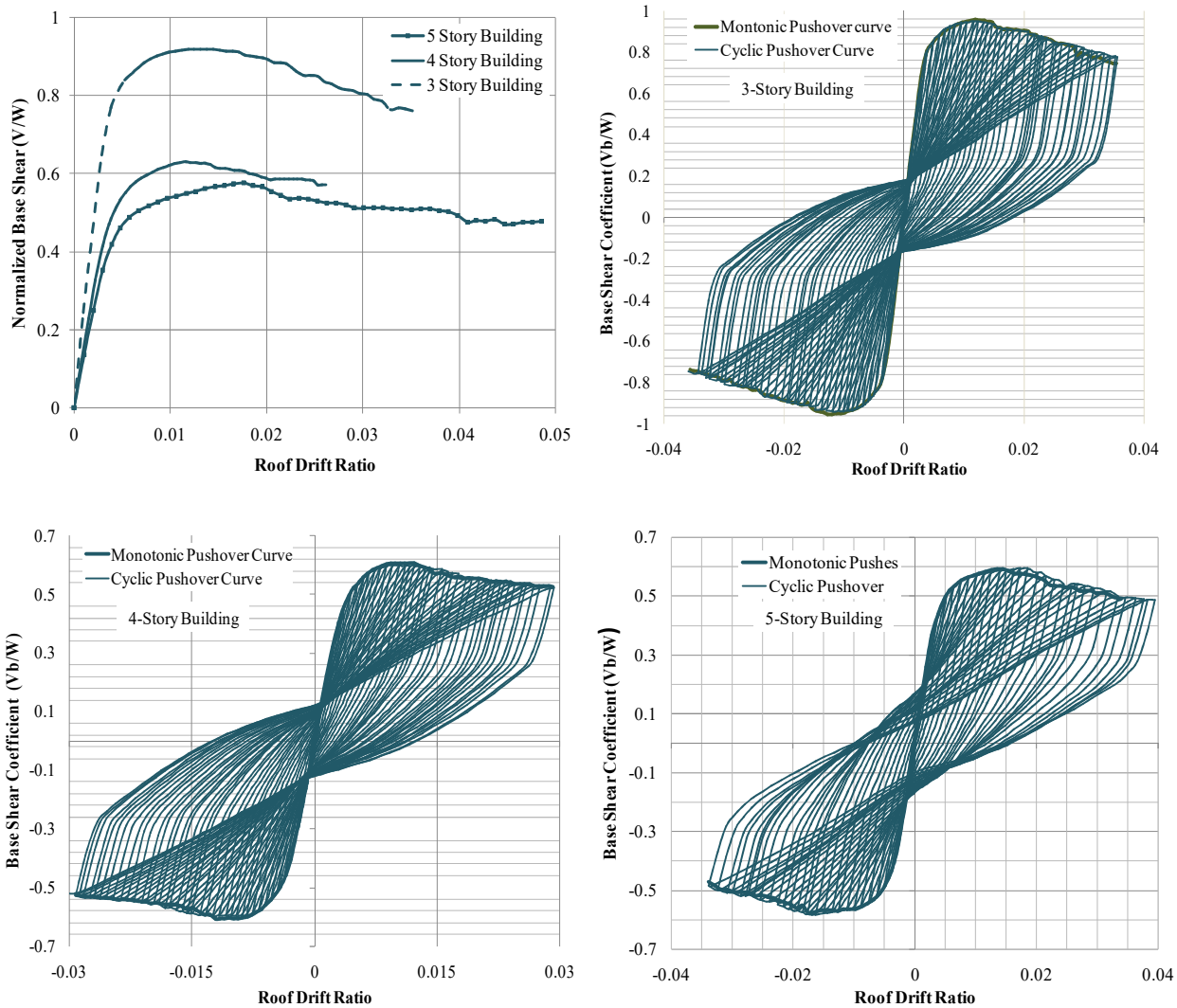


Fig. 8 The monotonic and cyclic pushover curves of three buildings for mode 1 in X-direction

The point-by-point application of equation (11) on cyclic MPA curves yielded hysteretic damping ξ_{hys}. The equivalent viscous damping ξ_{eq} is determined by adding the initial elastic damping ξ_{el} to hysteretic damping ξ_{hys} following the equation (10). Fig. 9 shows the equivalent damping ξ_{eq} versus roof drift ratio RD curves for the first mode in X-directions of case study buildings. Similar to the equivalent linear period T_e, a generalized equation to estimate the equivalent damping ξ_{eq} is proposed for low-rise masonry infill frames. The equivalent damping equation ξ_{eq} is as follows:

$$\xi_{eq} = \xi_{el} + \frac{0.2(RD_{Total} - RD')}{(RD_{Total} - RD') + 0.01} \tag{12}$$

Here ξ_{eq} represents equivalent viscous damping, and ξ_{el} is the initial inherent elastic damping of buildings. Similarly, RD_{Total} is the total roof drift ratio, and RD' is the roof drift ratio beyond which nonlinearity starts developing in the building. As a result, it will not be necessary to create a nonlinear model to obtain the cyclic MPA results to employ the SDBD procedure for masonry infill frame buildings.

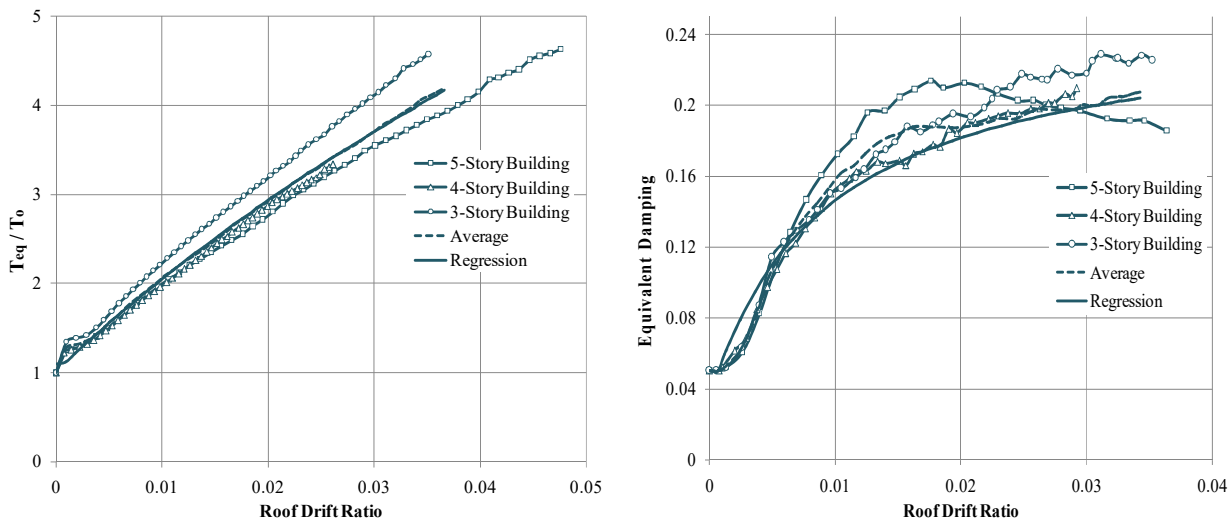


Fig. 9 Equivalent linear characteristics of case study buildings and generalized curves of infill frames

5. SELECTION OF GROUND MOTIONS:

The case study buildings, situated in Islamabad Capital Territory (ICT), are expected to experience a wide variety of seismic excitations in their intended design life. The ICT lies in the seismically active region of Pakistan and has faced several earthquakes of different magnitudes in past years. The two faults that mainly contribute to the seismicity of ICT are main boundary thrust and strike-slip faults (Zaman, 2016). Here, a suite of seven ground motion histories for these two fault types with magnitudes of $\mathrm{M}6.3 - 7.8$ are employed to carry out the detailed nonlinear time history analysis (NLTHA) procedure. These ground motion records are accessed using the NGA-West2 coast of PEER ground motion database and are most likely to occur within $10 - 50$kms of the source to site distance, with the site undergoing $490 - 620$m/s velocity of shear waves. Table 3 shows the details of these representative ground motions adopted to perform the NLTHA procedure. These time histories of earthquakes are adjusted and scaled by spectral matching in SEISMOMATCH software to match with the target response spectrum of UBC-97. This target spectrum is developed for a 5% damped design base earthquake (DBE) level for the region of ICT with $C_v = 0.32g$ and $C_a = 0.24g$ (UBC-97). Fig. 10 illustrates the matched response spectra of selected

ground motions with the target response spectrum. The case study buildings are expected to achieve various deformations and damage levels when subjected to these ground motions. Therefore, these earthquake records are considered suitable to evaluate the proposed SDBD procedure for low-rise infill RC frames.

Table 3 Details of selected ground motions

Event	Year	Station	Magnitude	Mechanism	R_{rup}(km)	V_{S30} (m/s)
Chichi Taiwan	1999	CHY010	7.62	Reverse oblique	19.96	538.69
Chichi Taiwan	1999	CHY029	7.62	Reverse oblique	12.65	573.04
Chichi Taiwan	1999	CHY087	7.62	Reverse oblique	28.91	505.2
Chichi Taiwan	1999	TCU042	7.62	Reverse oblique	26.31	578.98
Chichi Taiwan_06	1999	CHY029	6.3	Reverse	41.36	544.74
Chuetsu	2007	Nadachiku Joetsu city	6.8	Reverse	35.93	570.62
Iwate Japan	2008	Misato Akita City	6.9	Reverse	41.72	552.38

R_{rup}: The closest distance from the site to the earthquake rupture plane

V_{S30}: The average seismic shear wave velocity from the surface to a depth of 30 meters

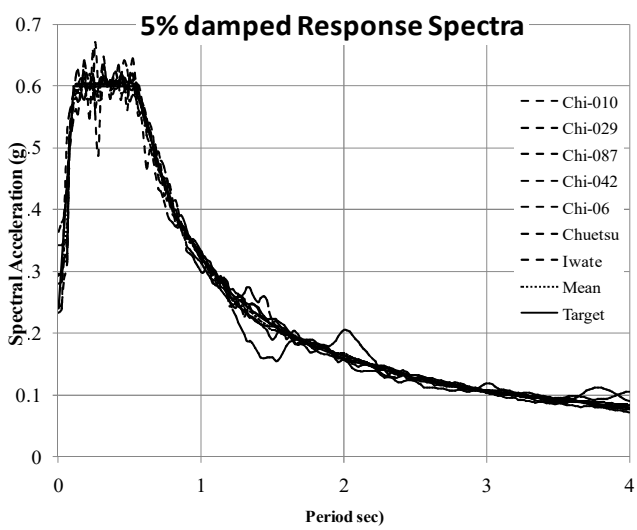


Fig. 10 The acceleration response spectra of selected ground motions matched with target

6. EVALUATION OF THE SDBD PROCEDURE

The SDBD procedure is used to determine the seismic demands of selected ground motions on a 5-story high building (B4). The most crucial step in the SDBD procedure is to accurately determine the equivalent linear properties of the building under consideration. Once the relationships of equivalent period T_{eq}/T_o versus roof drift ratio RD and equivalent damping ξ_{eq} versus roof drift ratio RD have been established, the equivalent linear characteristics of the building can be determined using the iterative approach of Section 2.2. In this suggested iterative scheme, the initial roof drift RD can be very small in some scenarios resulting in little additional damping and period elongation. In such cases, the seismic demands can be determined using the initial characteristics of the building.

In the SDBD procedure, a building model with elongated period T_{eq} is subjected to a displacement response spectrum, developed for an additional damping ratio ξ_{eq}. The seismic demands of the building corresponding to equivalent linear characteristics are estimated. The global responses (story shears and story moments) can be computed by applying the story forces on the respective floor levels. The structural model of building of equivalent period T_{eq} is developed by reducing the stiffness of frame elements and masonry infill panels simultaneously. The computed story forces are applied to the structural model of elongated period T_{eq} at the respective floor levels. The elastic analysis in any available commercial software will give you the other global responses (IDR profile and displacement profile) and the local responses of structural elements of the building.

Fig. 11 exhibits the comparison of global responses computed by the standard RSA method, the SDBD procedure, and the detailed NTLHA procedure for the case study building (B4). The results obtained from the NTLHA procedure are employed as a benchmark here. The seismic responses of low-rise buildings are generally governed by the 1st mode of vibration, so only the 1st mode for the case study building is considered here to determine its response. In the standard RSA method, a response modification factor R with a value of 3.5 is applied to reduce the seismic demands corresponding to the ordinary moment resisting concrete frame as per $UBC-97$. The inter-story drift ratios and displacements of the building are enhanced by the $0.7R$ factor to account for the inelastic behavior of the building.

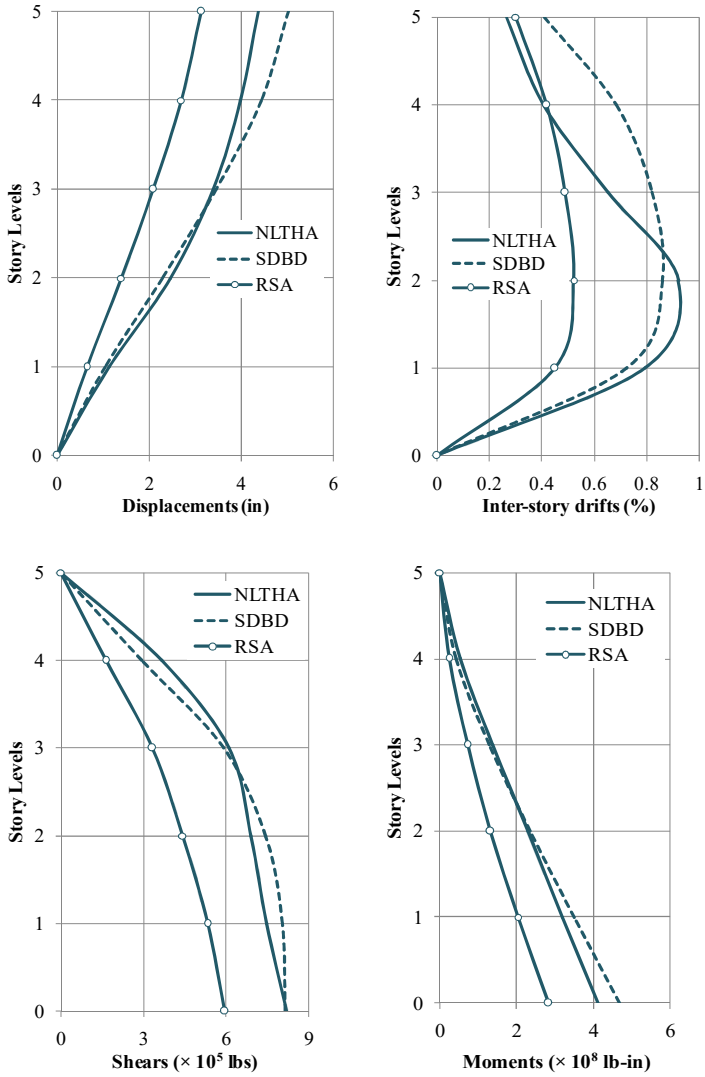


Fig. 11 The comparison of global responses between RSA, SDBD and NLTHA procedures

Fig. 11 indicates that story shears and story moments determined from the RSA method are considerably lower than the results of the NLTHA procedure for the overall height of the building. The inter-story drift ratio and story displacement profiles obtained from the RSA method are also on the lower side compared to the benchmark results of the NLTHA procedure. The reason for underestimating the seismic responses may lie in the choice of response modification factor *R* value. The factor *R* is the same for both the systems, with one system having low over-strength and high ductility while the other system with high over-strength and low ductility. However, studies (Seaoc, 2008) have exposed that the uncertainty in determining the seismic responses is primarily contributed by the ground motion histories because of their random nature. Therefore, a system with high ductility capacity is preferred over a system of high strength. It implies that a lower reduction factor should be assigned to high strength system

compared to the high ductile system. More studies and detailed research is needed in this regard because systems have become progressively more complex in terms of design, arrangements, and configurations.

On the other hand, the SDBD procedure is providing the estimations of global seismic responses in good agreement with the true demands predicted by the NLTHA procedure. The use of elongated period T_{eq} results in higher spectral displacement S_d than spectral displacement S_d at the initial period T_o for the target response spectrum under consideration. The global responses corresponding to the equivalent natural period T_{eq} are capturing the actual seismic demands better compared to the standard RSA method. However, a slight estimation error may be tolerated considering the simplicity of the SDBD procedure compared to the detailed NLTHA procedure. The respectable performance of the SDBD procedure demonstrated that it can be considered (and developed further) to determine the seismic demands of structures when it's not practical to perform the NLTHA procedure. The SDBD procedure offers the computational effort and convenience approximately similar to the linear elastic analysis with an additional step of determining the appropriate equivalent linear characteristics.

The member-level forces and deformations also play a vital role in the design and performance assessment of structural members besides the global responses of the building. The SDBD procedure can be employed to estimate the local response of members in any generalized structural analysis software because of its convenient application. Two columns, one exterior and the other interior, continued along the total height of the building, are adopted to evaluate the performance of the SDBD procedure. Fig. 12(a) displays the shear forces of two columns of a 5-story high building (B4), determined using the SDBD procedure, standard RSA method, and the detailed NLTHA procedure. It is apparent that shear force demands determined from the SDBD procedure are in good comparison with the NLTHA procedure, while the standard RSA method displays the enormous undervaluation of results. A similar utterance can be observed in the bending moment profiles of both columns, shown in Fig. 12(b). It is evident that the standard RSA method considerably underestimates the seismic demands, while the SDBD procedure captures the nonlinear demands closer to those obtained from the NLTHA procedure.

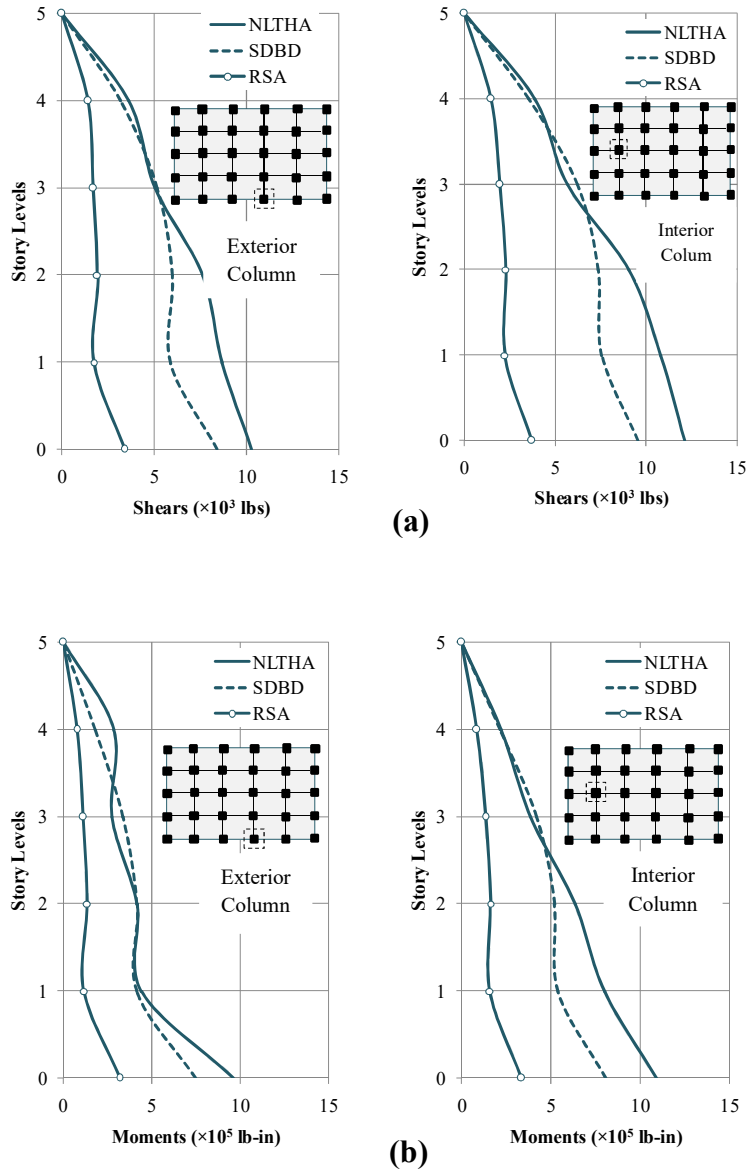


Fig. 12 The comparison of local responses among RSA, SDBD, and NLTHA procedures. (a) The shear forces of exterior and interior column along the total height of building. (b) The bending moments of exterior and interior column along the total height of building

The global seismic demands estimated by the SDBD procedure are satisfactory, but the member-level demands show notable differences compared to the true nonlinear demands of the NLTHA procedure. The difference may be due to the non-identical distribution of global seismic demands among the structural members. The comprehensive NLTHA procedure accounts for the actual distribution of seismic demands among structural members, while the SDBD procedure assumes their elastic distribution. Nevertheless, the member-level demands estimated by the SDBD procedure are considerably more appropriate than the demands determined from the standard RSA method.

Therefore, the SDBD procedure can be used to design new structures and evaluate the seismic performance of existing structures by predicting both local and global seismic responses with reasonable accuracy.

To implement the SDBD procedure in design offices, there's a need to develop generalized relationships between the equivalent linear characteristics of the system and appropriate deformation measure (e.g. roof drift ratio). The generalized relationships may incorporate extensive structural systems and hysteretic behaviors. The cluster of typical curves can be developed to determine the equivalent linear characteristics for general nonlinear behaviors of structures. This group of representative curves must cover the most practical ranges of governing hysteretic parameters (e.g. ratio of post-yield stiffness to initial elastic stiffness or area of hysteretic loops etc.). The developed relationships can be used conveniently to deal with numerous practical situations when applying the proposed SDBD procedure to structures of different details and configurations. Several graphic aids (e.g. group of displacement response spectra for different damping levels) can be provided to help the practicing designers to finalize the equivalent characteristics quickly through iterations. To accurately capture the response of high-rise buildings, the SDBD procedure can be modified further by incorporating the higher modes effects in its formulation.

7. CONCLUSIONS

This study proposes a simplified displacement-based design method using the framework of the DDBD method, based on the EL technique. The fundamental philosophy is that a suitably tuned linear elastic SDOF system having additional damping and elongated period can almost mimic the response of a nonlinear system. Therefore, a linear system can be appropriately used to determine the actual seismic demands of a nonlinear system. The best-known EL approach (determination of additional damping using equal energy principle at secant stiffness related to maximum displacement response) is employed to propose and evaluate the SDBD procedure for masonry infill RC frames. The results of a 5-story high case study building showed that the SDBD procedure can reasonably capture the local or global seismic response of infill frame buildings. The SDBD procedure neither requires nonlinear modeling nor nonlinear analysis to obtain the responses, so it keeps the simplicity offered by the linear elastic analysis. This study is merely an initiative in the direction of advancement in a more resourceful SDBD procedure. The application of other recognized EL approaches (i.e. instead of using the damping at secant stiffness) and proper distribution of global forces in structural members can enhance the accuracy of the SDBD procedure to predict the

seismic responses. Considering the effect of this concept on design offices, various improvements are needed in the SDBD procedure to make it suitable for building structures of different materials, designs, and configurations.

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Competing Interests

The authors have no relevant financial or non-financial interests to disclose.

Authors Contributions

All authors contributed to the study conception and design. Material preparation, data collection and analysis were performed by [Hamza Mazhar], [Musaddiq Afzal] and [Hassan Irfan]. [Fawad Ahmed Najam] was involved at each and every step of the work and all the work was done under his supervision. The first draft of the manuscript was written by [Hamza Mazhar] and all authors commented on previous versions of the manuscript. All authors read and approved the final manuscript.

Data Availability

The datasets generated during and/or analyzed during the current study are available from the corresponding author on reasonable request.