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Seismic performance evaluation of residential buildings in Dhaka city by using pushover analysis

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ABSTRACT: Bangladesh is situated in moderate earthquake prone region. Major metropolitan cities of our country are under serious threat because of faulty design and construction of structures. Weak buildings designed without seismic consideration could be vulnerable to damage even under low levels of ground shaking from distant earthquakes. So the structural engineers now-a-days are more concerned about the different earthquake analysis procedures. According to BNBC (2006) the buildings are designed according to equivalent static force method, response spectrum method and time history analysis. But the actual performance of a structure can be hardly found by these methods. Nonlinear inelastic pushover analysis provides a better understanding about the actual behavior of the struc-tures during earthquake. The pushover analysis which is not very familiar to many structural engineers has wide range of applications in the seismic evaluation and retrofit of structure. There are mainly two guidelines of this analysis-FEMA and ATC 40. The paper mainly follows the procedures of ATC 40 in evaluating the seismic performance of residential buildings in Dhaka. The present study investigates as well as compares the performances of bare, full in filed and soft ground storey buildings. For different loading conditions resem-bling the practical situations of Dhaka city, the performances of these structures are analyzed with the help of capacity curve, capacity spectrum, deflection, drift and seismic performance level. The performance of an in filled frame is found to be much better than a bare frame structure. It is seen that consideration of effect of the infill leads to significant change in the capacity. Investigation of buildings with soft storey shows that soft storey mechanism reduces the performance of the structure significantly and makes them most vulnerable type of construction in earthquake prone areas.

1 INTRODUCTION

Earthquake engineering has come a long way since its birth, and it seems to grow rapidly as we gain experience. Each time an earthquake happens, something new is available to learn and the profession grows to accommodate it. Both research and practice used to be mostly concerned with the design of structures that would be safe, in the sense of surviving a seismic event with minimum number of casualties. A structure designed to higher standards, chosen to meet the specific needs and able to remain functional after a small but relatively frequent event and being safe in a rare destructive earthquake costs slightly higher but still preferred now-a days by building owners. Pushover analysis is now considered most simplified inelastic analysis method to find actual behavior of the structure in earthquake, a powerful analysis method that would accurately analyze structural models and analyze the demand that any level of shaking may impose and specifically, determine the level of shaking that would cause a structure to exceed a specific limit-state, thus failing a given performance objective.

In last few years the widespread damage to RC building during earthquake generated greater demand for seismic evaluation and retrofitting of existing buildings in Dhaka. Furthermore, most of our buildings built in past two decades are seismically deficient because of the lack of awareness regarding structural behavior during earthquake and reluctance to follow the code guidelines. The structures, whose performances were evaluated in this study, are designed with the provisions from BNBC, 2006. BNBC equivalent static force method of determining earthquake force is limited to the structures having height of less than 20 meters. Hence this study deals with medium rise buildings (six-storied). The purpose of the paper is to summarize the basic concepts on which the pushover analysis is based, perform non linear static pushover analysis of medium height residential RC buildings as seen in Dhaka city and investigate the changes in structural behavior due to different infill configurations.

2 NONLINEAR STATIC PUSHOVER ANALYSIS

The pushover analysis of a structure is a static nonlinear analysis under permanent vertical loads and gradually increasing lateral loads. The equivalent static lateral loads approximately represent earthquake induced forces. A plot of the total base shear versus top displacement in a structure is obtained by this analysis that would indicate any premature failure or weakness. The analysis is carried out up to failure, thus it enables determination of collapse load and ductility capacity. On a building frame, plastic rotation is monitored, and lateral inelastic forces versus displacement response for the complete structure are analytically computed. This type of analysis enables weakness in the structure to be identified. The decision to retrofit can be taken in such studies.

Two key elements of a performance based design procedure are demand and capacity. Demand is a representation of the earthquake ground motion. Capacity is a representation of the structure's ability to resist the seismic demand .The performance is dependent on the manner that the capacity is able to handle the mand .In other words, the structure must have the capacity to resist the demands of the earthquake such that the performance of the structure is compatible with the objectives of the design. Once the capacity curve and demand displacement are defined, a performance check can be done. A performance check verifies that structural and nonstructural components are not damaged beyond the acceptable limit (Ellul & Ayala 2008) of the performance objective for the forces and displacements implied by the displacement demand.

In this study, non linear static pushover analysis was used to evaluate the seismic performance of the structures. The numerical analysis was done using ETABS 8.5.0 and guidelines of ATC-40 and FEMA 356 were followed. The overall performance evaluation was done using capacity curves, storey displacements and interstorey drift ratios. Plastic hinge hypotheses was used to capture the nonlinear behavior according to which plastic deformations are lumped on plastic hinges and the rest of the system shows linear elastic behavior (Li 1996). The Federal Emergency Management Agency in its report Pre standard and Commentary for the Seismic Rehabilitation of Buildings (FEMA-356, 2000) defines the structural performance level of a building to be selected from four discrete structural performance levels and two intermediate structural performance ranges. The discrete Structural Performance Levels are- Immediate Occupancy (S-l), Life Safety (S-3), Collapse Prevention (S-5), and Not Considered (S-6).The intermediate Structural Performance Ranges are the Damage Control Range (S-2) and the Limited Safety Range (S-4). The definition of these performance ranges are given by FEMA (FEMA-356, 2000).

3 MODELING OF MASONRY INFILL WALLS

In conventional analysis of infilled frame systems, the masonry infill wall is modeled using equivalent strut method or a refined continuum model. The former method is adopted in this particular paper for modeling of masonry infill. The approaches presented by Paulay and Pristlay (1992) and Angel et al.(1994) lead to a simplification in the infilled frame analysis by replacing the masonry infill with an equivalent compressive masonry strut as shown in Figure 1. They have assumed constant values for the strut width, 'a', between 12.5 to 25 percent of the diagonal dimension of the infill, with no regard for any infill or frame properties. Stafford-Smith and Carter (1969), Mainstone (1971), and others derived complex expressions to estimate the equivalent strut width, a, that consider parameters like the length of contact between the column/beam and the infill, as well as the relative stiffness of the infill to the frame. Expressions used here have been adopted from Mainstone (1971) and Stafford-Smith and Carter (1969), for their consistently accurate prediction of in-filled frame in plane behavior when compared with experimental results.

The equivalent strut width 'a', depends on the relative flexural stiffness of the infill to that of the columns of the confining frame. The relative infill-to-frame stiffness shall be evaluated using Eq. 1 (Stafford- Smith and Carter, 1969):

$$\lambda_1 H = = H \left[(E_m \, t \sin 2\theta) / (4 \, E_c \, I_{col} \, h_w)\right]^{1/4} \tag{1}$$

where t is the thickness of masonry wall.

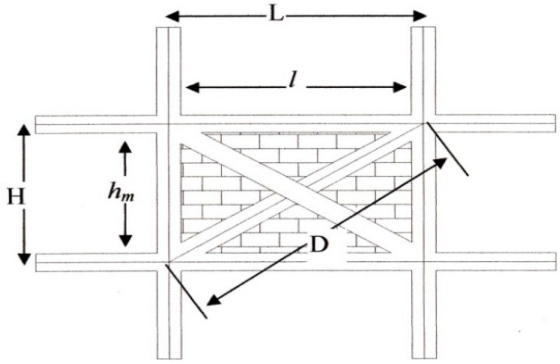


Figure 1. Strut geometry

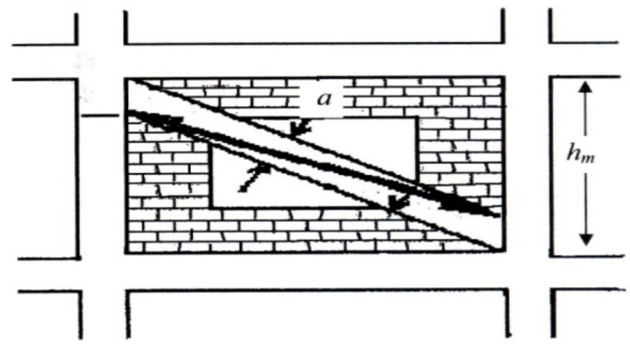


Figure 2. Placement of strut

Using this expression, Mainstone (1971) considers the relative infill-to-frame flexibility in the evaluation of the equivalent strut width of the panel as shown in Eq 2

$$a = 0.175D\,(\lambda_1 H)^{-0.4} \tag{2}$$

If there are opening present, existing infill damage, and/or FRP overlay, however, the equivalent strut must be modified using

$$A_{mod} = a\,(R_1)i(R_2)i\,\zeta_1 \tag{3}$$

where
$(R_1)i$= reduction factor for in-plane evaluation due to presence of openings
$(R_2)i$= reduction factor for in-plane evaluation due to existing infill damage
ζ_1= strength increase factor due to presence of FRP overlay

Although the expression for equivalent strut width given by Eq 3 was derived to represent the elastic stiffness of an infill panel, this document extended its use to determine the ultimate capacity of in filled structures. The strut was assigned strength parameter consistent with the properties of the infill it represents. A nonlinear static procedure commonly referred to as pushover analysis, was used to determine the capacity of the in filled structure. The equivalent masonry strut is to be connected to the frame members as depicted in Figure 2., where the bold double sided arrow represents the location of the strut in the structural model. The infill forces are assumed to be mainly resisted by the columns, and the struts are placed accordingly. The strut should be pin connected to the column at a distance l_{column} from the face of the beam. This distance is defined in Eq 3 and Eq 5 and is calculated using the strut width, a, without any reduction factors.

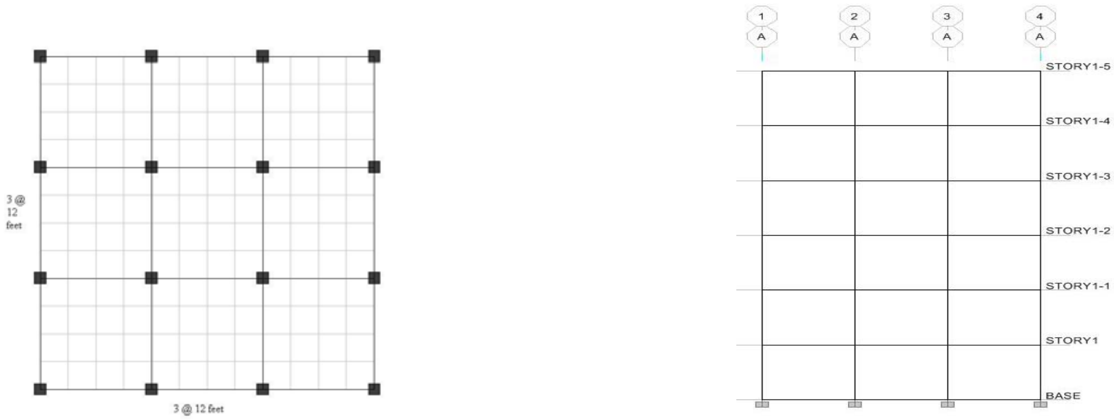
$$l_{column} = a/\cos\theta_{column} \tag{4}$$

$$\tan\theta_{column} = \{h_m - (a/\cos\theta_{column})\}/l \tag{5}$$

Using this convention, the strut force is applied directly to the column at the edge of its equivalent strut width. Figure 2 illustrates these concepts. Modulus of elasticity of the masonry units was chosen considering the ACI/ASCE/TMS masonry code as 1200 Ksi .

4 DESCRIPTION OF THE STRUCTURAL FRAME

For analytical application, a sample RC three dimensional building is selected. In order to concentrate on the effects caused by the distribution of infill the prototype bare frame structure is regular throughout its height and bay length. The structure is six storeys high, with a storey height of 3 meters (Fig. 3). The bay lengths are 3.7m-3.7m-3.7m in both directions. The slab thickness is 125 mm and the column sizes are 500x500 mm for

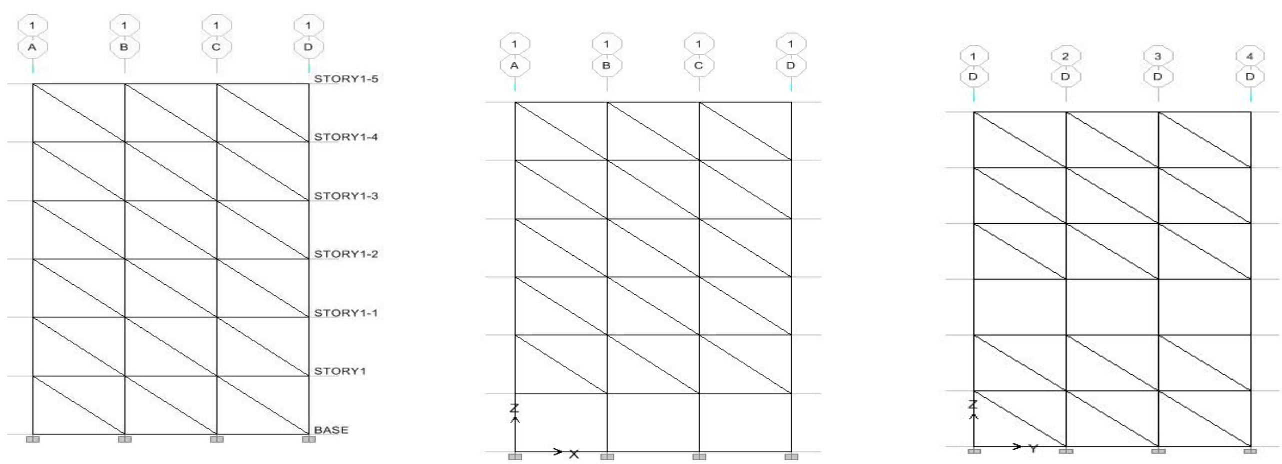


(a) Plan (b) Elevation

Figure 3. Plan and Elevation of the prototype bare frame structure

interior columns and 400x400 mm for exterior column. All the beams are of the same size with a width of 300mm and depth including slab thickness of 500 mm. The building is as assumed to have mat foundation. The concrete strength was assumed to be 4000 psi with yield strength of steel to be 60000 psi. Young's modulus of elasticity for concrete is 3600 Ksi .As regards the masonry infills; they were modeled as equivalent diagonal strut, as mentioned earlier, with a width of 485 mm and thickness of 125 mm. The masonry infill considered is representative of a weak masonry having a compressive strength of 1 MPa. The sample structure is assumed to be located in Dhaka, which belongs to the seismic zone 2 according to BNBC (2006). Considering standard occupancy structure and exposure category A, equivalent earthquake loads are determined. The geometry and the material characteristic together with the fact that the infill is in direct contact with the fact reflect common practices of Bangladesh were infilled frames are not engineered to resist earthquakes.

In order to investigate the effect of infill distribution three different geometrical possibilities were explored (Fig. 4). The first case comprises a fully infilled structure resembling the regular structures thus representing a regular distribution of stiffness throughout the height. On the other hand the second case examined the effects of omitting infills from ground floor only, such as with the infamous soft ground storey configuration. On the other hand the third case specifically dealt with the consequences of omitting the infills of the third floor of the building and observed the influences on structural performances. The load deformation responses of the numerical specimens were followed through to failure by means of the capacity curve. The curve was established using non linear static pushover analysis, wherein the loading profile used was a triangular one commensurate to the dominant first mode distribution of the seismic loads. In order to assess the seismic performance, moment hinges (M3) were assigned to both ends of beams and axial hinges (P-M-M) were assigned to the column ends. Geometric non linearity (P-Δ) and large displacement is considered with full dead load and when local hinges fail redistribution of loads is allowed by unloading



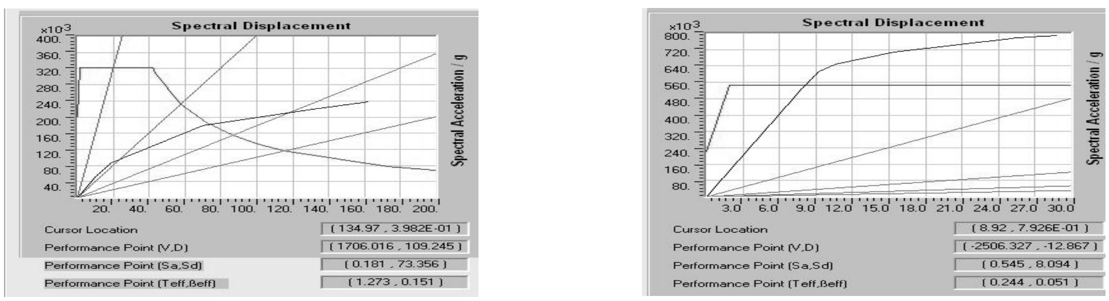
(a)Full Infilled Frame (b) Soft ground storey (c) irregular structure

Figure 4. Different infill configurations used in analysis

entire structure. The gravity loads used for analysis included self weight of the members and loads of floor finish and live loads were applied according to BNBC. All partition walls were assumed to be located directly on beams. The performance points marked by collapse and representing ultimate displacement capacity of the structure were evaluated at each step of the analysis according to the guidelines of ATC-40 and FEMA 356.

5. COMPARISON OF STRUCTURAL PERFORMANCE

The relative performance of each configuration is reviewed by comparing the performance curves for each case (Fig. 5). On comparing the behavior of fully infilled frame against the behavior of the bare frame, it was seen that the bare structure had to deform a considerable amount before the capacity curve meets the demand curve. As a result some of its structural elements were stressed above their elastic limit but for the infilled frame the demand and capacity curves intersected within the elastic region. So the structure did not show significant damage at performance point.



(a) Bare Frame (b) Full Infilled Frame

Figure 5. Comparison of Capacity Curves of bare and full infilled frame

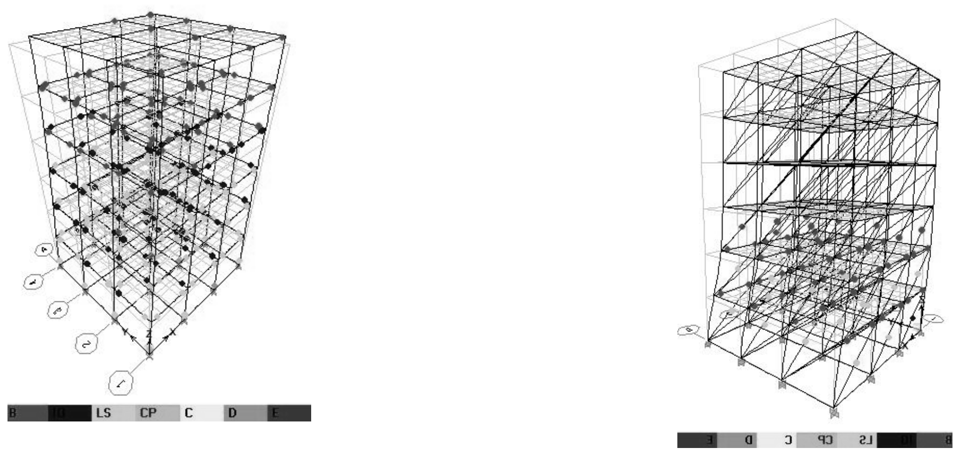
For easier understanding, hinge conditions of the two structures are represented in Figure 6. The stepwise analysis of deformed shapes in pushover analysis made it evident that plastic hinge formation was initiated in the masonry walls in the infilled frame with subsequent hinge formation in the beams at the performance level. This is contrary to the performance of the bare frame which suggests an increased seismic vulnerability of the structure with formation of column hinges at base level and beam hinges at each storey level at the performance point (Fig. 6). The Infilled frame with few partition walls reaching immediate occupancy and the whole upper portion remaining intact definitely indicate a better seismic performance. The Table 1 gives a detail list of number of hinges formed including their type. It shows that the infilled structure gave much better performance with only 23 hinges above LS level and all of them on the partition walls indicating negligible damage to the main structure.

The difference in behavior between the two is further appreciated by referring to Figure 7, Figure 8 and Figure 9, depicting the base shear, inter-storey drifts and lateral displacement. The gain in initial stiffness resulting from addition of partition wall increased the base shear taking capacity of the structure as shown in Figure.7, about 1.5 times. The performance of the full infilled frame improved significantly both in terms of inter-storey drift and lateral displacements as shown in Figure 8 and Figure 9. The maximum story drift and displacements as observed for infilled frame were decreased by about 75 percent and 86 percent respectively, compared to the bare frame.

On the other hand significant differences in behavior were noticed for the soft storey structure, which set out to identify change in behavior due to significant change in stiffness between adjacent stories. As expected the soft ground storey structure resulted in a much reduced tolerable base shear capacity, as shown in Figure 10, which is no less than 25 percent. Furthermore formation of column hinges at performance point as shown in Figure 12, including some hinges above LS-CP level is also noticeable. From Figure 9, which details the inter-storey drifts for predicted collapse, an exceptionally high drift was observed at storey 1 for the soft storey structure which suggests an increased vulnerability for the upper floors, with collapse at this level being registered prior to that in the ground floor. With the relatively large value of displacement observed this configuration which is most popular in residential apartment buildings of Dhaka, proved to be highly detrimental to the overall performance of the structure.

Table 1: Number of hinges formed at performance point

Structure	A-B	B-IO	IO-LS	LS-CP	CP-C	C-D	D-E	>E	Total
Bare	263	80	83	100	0	0	2	0	528
In-filled	588	60	1	22	0	1	0	0	672
Soft storey	534	34	40	36	0	4	0	0	648



(a) Bare Frame (b) Full infilled

Figure 6. Comparison of deformed shapes at performance point showing formation of plastic hinges

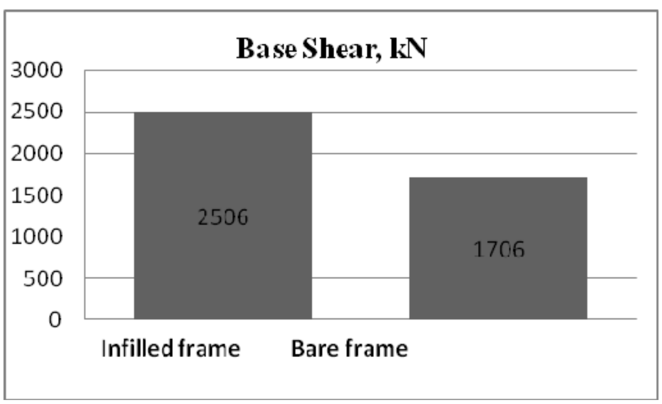


Figure 7. Comparison of Base Shear

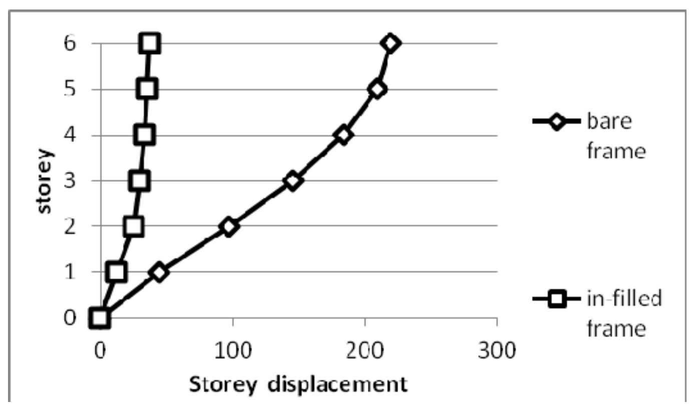


Figure 8. Comparison of storey displacement

The possible formation of a weak story by omitting the infills of storey 3 was analyzed as a special case. These types of irregular structures are most common in apartment buildings where a particular floor is used as convention center. Consequently when compared to the full infilled structure it showed reduction in capacity and large displacement resembling the behavior of a soft storey structure. The maximum tolerable base shear for the structure shown in Figure 10 is 3193 kN which is only 75 percent of the full infilled structure. The storey displacement as illustrated in Figure 11, changes abruptly from 5.7mm to 17mm for the empty floor and the story drift is also the maximum at that floor. The step wise analysis of ETABS showed that structural damage initiated at that particular floor and at performance point the beams below the floor were affected the most. Interestingly the configuration did not result in formation of column hinges like the soft ground story structure, however a major increase in demand on that floor was observed. Overall the change in stiffness induced by the different infill configurations proved to be very damaging with both the configuration registering significant decrease in ultimate capacity compared to the fully infilled case.

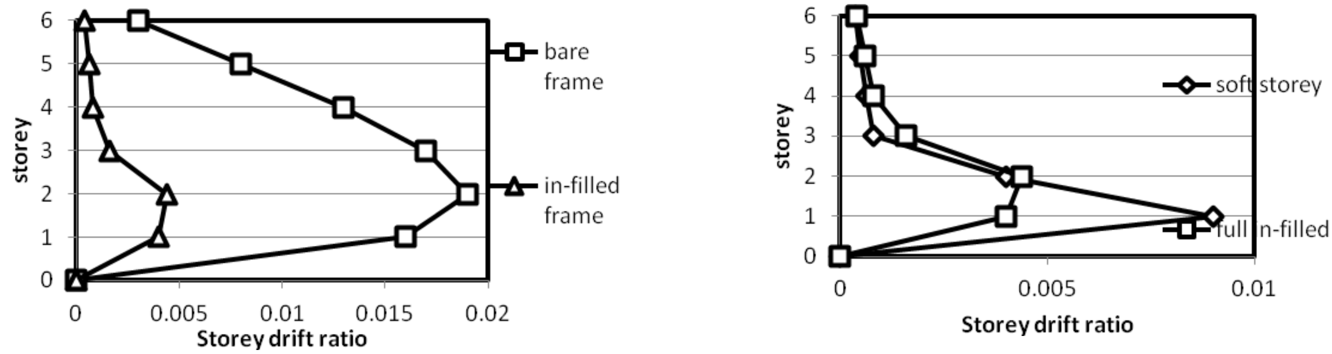


Figure 9. Comparison of story drift

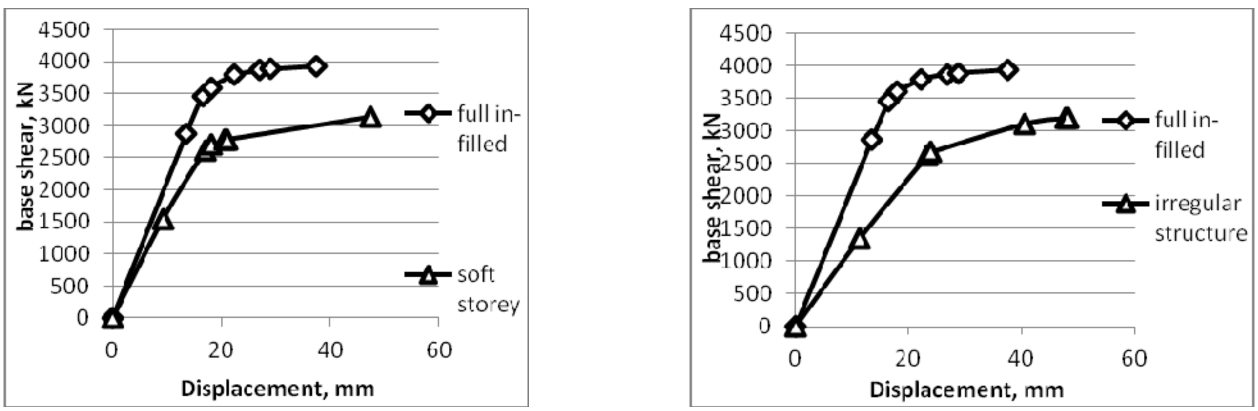


Figure 10. Comparison of base shear

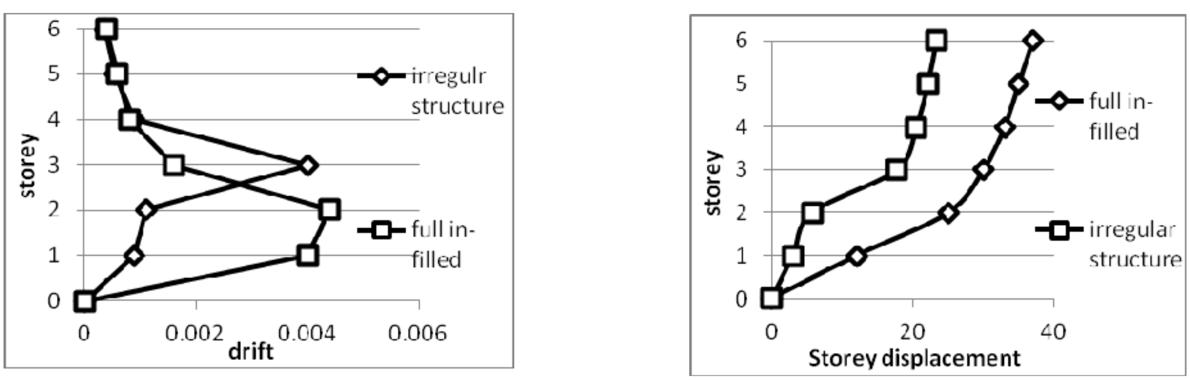
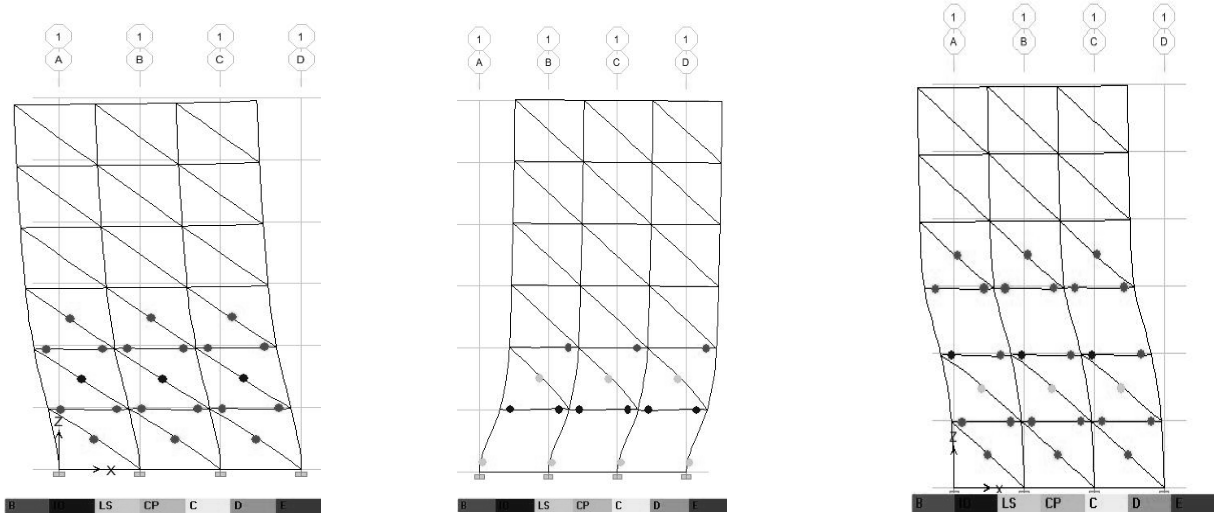


Figure 11. Comparison of storey drift and storey displacement



Full infilled Soft ground story Irregular structure

Figure 12. Comparison of deformed shapes at performance point showing formation of plastic hinges

6. CONCLUSION

The parametric analyses conducted in the present study highlighted that masonry infill distribution have significant influence on the performance of the structure. Certain configuration proved to be highly detrimental and possessed unique characteristics. Nevertheless, it has been shown that the fully infilled frame undoubtedly outperformed the bare frame structure and had improved ductility. Conversely the soft ground storey mechanism proved to be highly unfavorable. In both of the cases of soft storey analyzed here, structural damage was initiated on that storey and propagated rapidly. However a delicate balance between improvement and degradation was observed with little alteration of infill position, which brings us to a conclusion that the provision of infill in not always beneficial or always detrimental, rather depends on the configuration and how it changes building stiffness locally. But in either case the necessity of modeling the infill is inevitable as it enables the designer not only to have a better understanding of the structural response but also predict the potential zones of local failure. It is highly advised that the designer while omitting partition walls of any particular floor would make necessary adjustments to the surrounding structural members and strengthen them accordingly.

The fact that Dhaka, the highly populated metropolitan city densely crowded with medium to high rise RC buildings, is frequently facing earthquakes of low to medium intensity and expecting some serious seismic threats in the near future, emphasizes the importance of using an appropriate numerical model such as one presented in this study for the actual seismic assessment of the RC constructions. There are good reasons for advocating the use of the inelastic pushover analysis for demand prediction, since in many cases it will provide much more relevant information that an elastic static or even dynamic analysis and encourage the design engineer to recognize important seismic response quantities and to use them for exposing design weaknesses.

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