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Performance based comparative analysis of bare frame and infill frame analysis of typical two story building in Nepal

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Abstract: Structural retrofitting is especially carried out for strengthening the existing building so that it would withstand the seismic load without collapse. But many recent earthquakes have shown that though the building withstands the earthquake without major damages, it may not be useable for immediate occupancy. Besides, the fallen walls can cause serious accidents threatening life safety. Though new building codes in different parts of the world have tried to minimize such deficiencies by including different detailing requirement, the advantages of those detailing cannot be quantified properly. So, performance based analysis is becoming popular among engineers as it could satisfy the client with the desired level of safety. Recently, many local authorities have shifted to performance based design from traditional strength based design for the retrofitting purpose. This paper mainly focuses on performance based comparative analysis of typical two storied building in high seismic zone of Nepal. As performance based retrofitting is new in Nepal some of the common modeling errors which are commonly adopted by the designers are highlighted and proper methods are suggested.

Keywords: **Performance based design; Retrofitting; Typical Building; Bare frame model; Infill wall model.**

I. INTRODUCTION

Structural retrofitting is one Nepal lies in the earthquake fault dividing Indian and Eurasian Plates. A total of 92 active faults have been mapped throughout the country by the Seismic Hazard Mapping and Risk Assessment [1]. So, it is a seismically active country and ranked as 11th seismically vulnerable country in the world [2]. The earthquake history shows that strong earthquakes (≈Mw 8) had reoccurred around every 80 years [3]. Recently on April 25, 2015, Nepal was hit by a strong earthquake of magnitude Mw 7.9 which was given name “Gorkha earthquake”. It was found that eleven out of sixteenapartments in the capital city Kathmandu where not suitable for the immediate occupancy (IO)[3] though they were claimed to be designed according to the prevailing (strength based) building code in the country. So, engineers and government authorities in Nepal are now shifting towards the performance based design from traditional strength based design. Especially, the local authorities demands the performance based analysis and design for the building retrofitting.

This paper present the performance based evaluation and design of hospital building in Nepal. As moment resisting RCC (Reinforced Cement Concrete) structures are very common in Nepal for building construction [4] most of the hospital buildings are RCC structure. Recently, researchers have done a lot of work in RCC structures. Different advanced techniques have been already developed [5-10]. The most of the retrofitting works in Nepal are usually based on traditional retrofitting techniques like column-beam jacketing and adding concrete wall. So, this paper is also based on the column and beam jacketing technique.

Though much work have been done in retrofitting many researchers still prefers to neglect the effect of masonry infill in the contribution of base shear capacity of structures[11,12]. Experiment conducted by Pujol and Fick [13] has shown that masonry infill wall can increase the base shear capacity around 100% and stiffness of the structure around 500%. Hence, in this paper, the effect of masonry infill was captured by commonly accepted strut.

II. BUILDING DESCRIPTION

The elevation and plan of the building is shown in Fig 1. It is a small office building with two story. These types of building structures are common in capital city Kathmandu as the soil condition is not so good in Kathmandu. The properties of the building components are given in Table 1. It is reinforced concrete cement (RCC) building.

Loads of each floor slab are transferred to columns by beams. Columns transfer their loads to ground by pad footings.

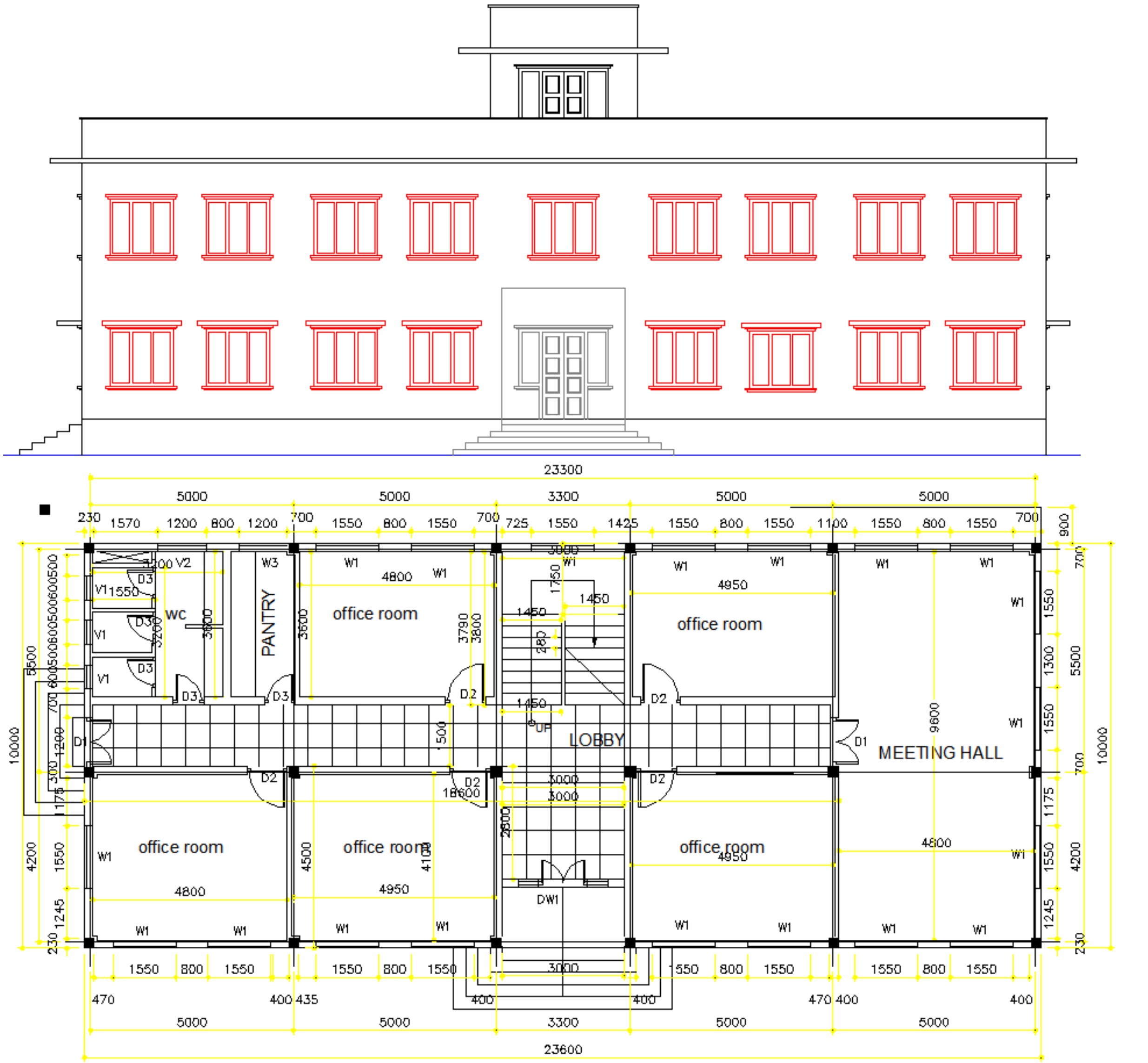

Fig 1. Elevation and Plan of two story building

Table 1. Structural properties of building

Pedestal height =	0.9	m
Floor height =	3.2	m
Height of pent house on roof =	3.2	m
Building height, h =	10.5	m
Length =	23.6	m
Width =	10	m
Steel grade, fy =	415	MPa
Concrete grade, fck =	15	MPa

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All column sizes are 9” × 9” with four 12 mm diameter rebar and 250mm center to center spacing of stirrups (6ϕ) but the finished size of the columns are 10”×10”. Beam sizes are 9” in width and 14” in height with same rebar number and size as that of column. Ground floor slab was not attached to the structure. Slabs of 150 mm thickness were at the first and second floor level.

III. MODELING THE BUILDING

The building was modelled with SAP 2000 v 18.0.1. Beams and columns were modelled with line element while slab was modelled with thin plate element. Pedestals below the tie beams were fixed at the level of pad footing. Suitable material properties are assigned from the available materials in the material library. Slabs were discretized in 1m x 1m pieces to get the better result. Dead loads were assigned as wall load and self-weight of structure while live load were assigned following Nepal Building Code (NBC 103:1994) [14].

Nonlinear static pushover (NLSPO) analysis was used for the performance analysis. First the building was checked with the bare frame modeling and its fundamental period of vibration was checked with the approximate period prescribed by the code $T_1 = 0.09h/\sqrt{D}$ for infill structures where T1 is the fundamental period, h is the height to structure in meter and D is the width of the structure in considered direction. The software generate fundamental period was 0.91s while the calculated period from the code was 0.26s. Such discrepancy can be controlled in SAP 2000 for equivalent lateral load procedure of seismic analysis, directly inputting the fundamental time period by user so that the software itself considers the higher base shear demand. But in case of pushover analysis, fundamental period of the structure cannot be input directly and software generates the demand and capacity point (performance point) based on the software generated period. So, only one way of tackling the problem is to model the structure correctly to account for the effect of masonry infill. So, masonry infill was modelled with different approaches [15, 16]. Strut model given by Paulay and Priestley [15] was found to be more suitable in terms of the structure’s time period as it gave0.29s which is very near to the period given by code for infill frame structure. The young’s modulus E for brick strut was assumed to be 1750 MPa and compressive strength is taken to be 1.75 MPa (0.25f’m, f’m = 7MPa, for second class brick) which is generally adopted values for second class brick in Nepal. Masonry openings were considered to be 15% at the wall center for the strut modeling and the opening coefficient is adopted according to Asteris et al. [16].

The building was analyzed with NLSPO method. Initially, the base shear demand was not met by the structure. The beam and column elements which failed first where separated in group and were reanalyzed by jacketing. Since the building was just of 2 story, the column of size 14”x14” and beam of size 14”x18” (width × height) were assumed to be the starting point of jacketing. This process was repeated until the building’s members fulfils the requirement of immediate occupancy (IO) for serviceability earthquake, Life safety (LS) for design earthquake and Collapse prevention (CP) for the maximum considered earthquake. It should be noted that serviceability earthquake was considered to be half of design earthquake and maximum considered earthquake is 1.25 times the design earthquake in intensity. Normally design earthquake is considered to be 1.5 times of design earthquake but the building was considered to be in zone 4 of ATC – 40 [17] whichpermits to take 1.25 factor for maximum considered earthquake. The rotation limits of the members are adopted according to FEMA 356 [18]. ATC – 40 [17] recommends the use of a set of seven or more pair of time history componentfor the design to be based on the average value of the response quantity of interest and at least three pairs of horizontal time history components should be selected from earthquake groundmotion records. But the required number of time history is not available for Nepal so the response spectrum was constructed following ATC – 40 [17] selecting the similar zone factor Z = 0.4, near source factor, NA =1, Nv =1 and seismic source type A. The adopted values for Ca and Cv for the design earthquake are 0.44 and 0.64 respectively.Ca and Cv are the coefficient by which demand response spectra are constructed. According to ATC -40 [17], 2.5gCa is considered as the maximum peak value of spectral acceleration up to upper limit period Ts = Cv/2.5Ca second while TA (=0.2Ts in second) is the lower limit period (Fig 2).

During analysis it was found that infill wall only contributes in the early stages of pushover as its strength was comparatively very low, so, its strength was neglected during analysis.But the challenge was to maintain the fundamental period so that the model would incorporate the effect of stiffness due to masonry because normally higher stiffness demand higher base shear capacity. The time period of the structure was then check after retrofitting which was found to be 0.42s for bare frame and 0.33 s with infill masonry provided that frame elements are same for both. The time periods were checked whether they lie in the plateau (maximum spectral

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acceleration) region or not according to ATC -40 [17]. The upper limit of time period was found to be Ts = 0.58s and lower limit was found to be TA = 0.11s, thus both the periods (0.48s and 0.33s) are in the plateau region which demands maximum base shear. So, no correction was needed for the stiffness provided by infill masonry.

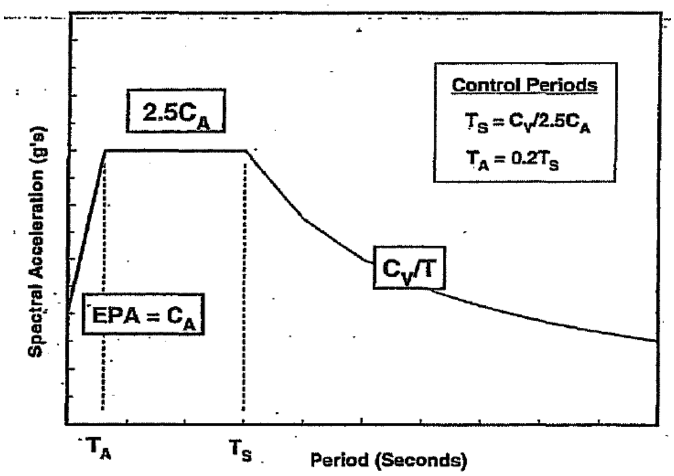
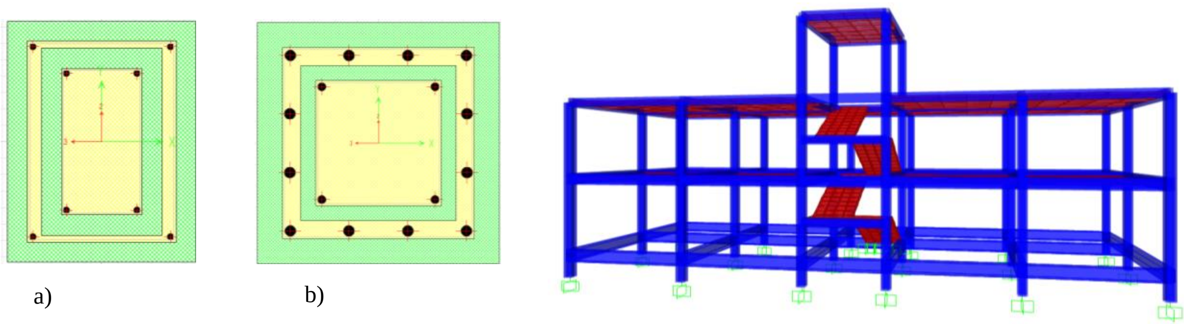


Fig 2. Construction of 5% damped elastic response spectrum [17]

Though bare frame fulfills the rotation requirement of ATC-40 [17] and FEMA 356 [18] for IO, LS and CP, it was found that masonry failed in the IO level of frame demanding the need of strengthening the masonry wall. Sothe walls were suggested to reinforce by attaching wire mesh which could prevent the sudden falling of wall in case of failure. Wall strengthening or reinforcing is normally not included in retrofitting because they are not normally modelled along with frame structures as a result designer are unaware of their performance during the earthquake. Typical frame section with jacketing and SAP 2000 model of the building is shown in Fig 3.



a) Typical retrofitted beam: Original/inner size 9”x 14”(width x height): Retrofitted beam 14” x 18”
b) Typical retrofitted Column: Original/inner size 9”x 9”(width x height): Retrofitted beam 14” x 14”
c) SAP 2000 model of the building

Fig 3. Typical retrofitted sections and SAP 2000 building model

IV. RESULT AND CONCLUSION

Analysis result of the frame structure with and without infill walls shows that there is significant influence of the infill in the structure as the extra base shear contribution of the infill walls in x and y directions are 133% and 16% respectively which is in accordance with the experiment conducted by Fick [13].But as the push over progresses the infill masonry cannot provide strength and stiffness due to sudden brittle failure (Fig 4). It means there is no influence of masonry infill during LS and CP stages of bare frame. So, it is not recommended to include masonry infill strength during analysis. But it is recommended to analyze the masonry and provide strengthening/retrofittingif needed.

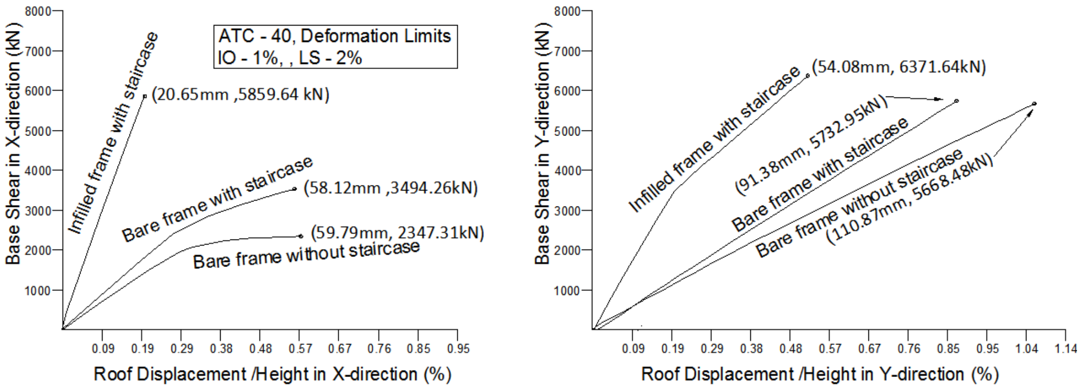


Fig 4. Roof displacement versus roof displacement with and without infill masonry

Besides, infill modeling can also contribute to check the structure period in different modes of vibration which would help the designer to decide whether the stiffness considered in bare frame is adequate or not for the design purpose.

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