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Evaluation of Overstrength Factor and Ductility Factor for Masonry Infilled Steel Frame Buildings

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Abstract
Steel framed buildings has a growing popularity in Nepal due to its faster construction, availability of material and availability of manpower. Yet it is common to see masonry as an infill to create partitions and external cladding because of its good functionality, cheap construction and little expertise. Masonry infill present in these structures however increases the stiffness in a global aspect which leads to shorter time period and smaller drift. The over strength factor and ductility factor, which depends upon the time period and drift of the structure, are hence affected due to this. In this study, steel framing buildings are investigated with regards to their over strength and ductility incorporating the effect of infill. Number of frame layouts with and without the effect of infill are studied and evaluated. Steel frames are designed and analyzed as per IS 800: 2007 and the seismic code used is NBC 105: 2019. Nonlinear analysis is carried out as per available literature and provisions in FEMA356:2000.

Keywords
Brick Masonry Infill, Steel Frame, Overstrength Factor, Ductility Factor, Response Reduction Factor

1. Introduction

Steel frame structures are the ones in which the framing elements are made of structural steel. Brick masonry is one of the most economical building materials in construction works. Steel frame buildings with masonry infill has a growing popularity in Nepal and has been used in residential, industrial as well as commercial buildings. As the construction process is faster with steel structures and masonry infill provides good functionality with little expertise, steel framed buildings with masonry infill will most likely be seen often in the coming decade.

It is a common practice to consider the infill as a nonstructural element during the design and analysis of the structure. Masonry infill increases the stiffness and strength in a global aspect leading to a stiffer frame which results to a shorter time period and smaller drift. As over strength factor and ductility factor depends upon the time period and the drift of the structure, the effect of masonry infill needs to be studied for it. Utilizing the inelastic behavior can however bring advantage in lowering the construction cost by reducing the size of members. To do this, the main approximation lies in the concept of response reduction factor (R) which depend upon the overstrength and the ductility of the members.

2. Overstrength and Ductility Factor

In the mid-1980s, data from an experimental research program at the University of California at Berkeley were used by the Berkeley researchers to describe a Reduction factor R as the product of three factors to account the reserve strength, ductility and viscous damping.

$$R = R_s.R_\mu.R_d \tag{1}$$

where R_s is the over strength factor (to quantify design overstrength, material overstrength and system overstrength), R_μ is the ductility factor(which accounts for global nonlinear response of a structure) and R_d is a damping factor to account the damping in the framing system. The damping factor is usually neglected by codes unless the structures have supplemental damping devices. A separate factor relating to Redudancy R_ζ is added by the studies conducted by Applied Technology Council.

$$R = R_s.R_\mu.R_\zeta \tag{2}$$

The over strength factor can be calculated easily by approximating a bilinear curve from the pushover curve of the structure to obtain the yield strength V_y and taking the ratio with design base shear V_d.

$$R_s = \frac{V_y}{V_d} \tag{3}$$

However, ductility factor requires further calculation even after obtaining the bilinear curve.

The extend of inelastic deformation experienced by the structural system subjected to a given ground motion or a lateral loading is given by the displacement ductility ratio,μ, which is defined as the ratio of maximum absolute relative displacement to its yield displacement.

$$\mu = \frac{u_{max}}{u_y} \tag{4}$$

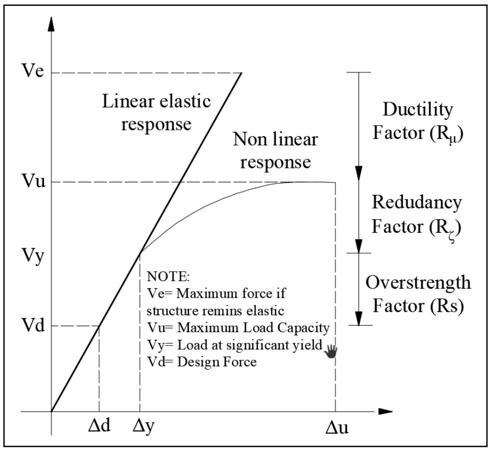

Figure 1: Concept of Response Reduction Factor

The ductility reduction factor is the ratio of Fy(μ=1), which is the lateral yield strength required to maintain the system elastic, to the Fy(μ=μ_i), which is the lateral yield strength required to maintain the displacement ductility ratio u less or equal to a predetermined target ductility ratio.

$$R_\mu = \frac{V_e}{V_y} = \frac{Fy(\mu = 1)}{Fy(\mu = \mu_i)} \tag{5}$$

Some factors that influence R_μ other than μ also includes the period of vibration, local soil condition, magnitude, epicentral distance, hysteretic behavior and damping. However, R_μ is significantly more influenced by displacement ductility ratio, period of vibration and the soil condition. The approximate force reduction factor Ru is given by

$$R_\mu = f(\mu, T, SC) \tag{6}$$

Where SC= Soil Condition

Regardless of the soil condition, the expression 5 has to satisfy the following condition:

$$\lim_{T\to 0} R_\mu = \lim_{T\to 0} f(\mu, T, SC) = 1$$

$$\lim_{T\to +\infty} R_\mu = \lim_{T\to +\infty} f(\mu, T, SC) = \mu$$

$$R_\mu = f(\mu, T, SC) = 1; \qquad \mu \leq 1 \tag{7}$$

The equation for R_μ for medium soil developed by Miranda and Bertero derived using detailed study of 124 ground motions recorded on a spectrum of soil condition has been used to determine R_μ in this study which is as shown in the following equation:

$$R_\mu = \frac{\mu - 1}{\phi} + 1 \geq 1 \tag{8}$$

Where ϕ=1+$\frac{1}{12T-\mu T}$-$\frac{2}{5T}$ exp[-2(lnT-$\frac{1}{5}$)^2]

The seismic codes in most of the countries have a reduction factor to reduce the elastic force to the design earthquake force. Some of the seismic codes have been listed below:

- Nepal- NBC 105: 2019- Overstrength factor and Ductility factor- Design dependent on Overstrength and Ductility
- Nepal- NBC 105: 1994- Structural Performance factor- Design dependent on ductility
- India- IS 1893(Part I): 2016- Response Reduction factor-Design dependent on overstrength, ductility and redundancy factor
- Bangladesh- Bangladesh National Building Code 2015- Response Reduction factor, R-Design dependent on overstrength, ductility and redundancy factor
- Pakistan- Building Code of Pakistan- Seismic Provision 2007- Numerical coeffecent representative of the inherent overstrength and global ductility capacity, R-Design dependent on overstrength, ductility and redundancy factor
- Chile- NCh433.Of96- Response Modification factor, R or R_o- Design dependent on energy absorption and dissipation characteristics of structure.
- USA- ASCE-7: 2016- Response Modification Coefficent, R-Design dependent on overstrength, ductility and redundancy factor

- New Zealand- NZS 1170.5: 2004- Structural Performance factor S_p- Design dependent of design on overstrength, ductility and redudancy factor
- Europe- BS EN 1998-1: 2004- Behavior factor, q-Design dependent on overstrength, ductility and redundancy factor

3. Mathematical Model of Infill

The infills in the building is represented by an equivalent strut attached at the diagonal ends of the infill. The material property of the equivalent strut will be same as that of the infill whereas the geometry will be calculated based upon various available literature. The width of the equivalent strut, α, depends upon the relative stiffness of the infill to that of the columns confining frame. The relative stiffness will be evaluated based upon the relation provided by Stafford- Smith and Carter, 1969

$$\lambda_1 H = H[\frac{E_m t sin2\theta}{4E_c I_{Col} h}]^{1/4} \tag{9}$$

Where, H is the height of column between centerlines of beams, E_m is the Modulus of Elasticity of Masonry Prism, t is the thickness of infill, E_c is the Modulus of Elasticity of frame material, I_{Col} is the moment of inertia of column, h is the height of infill panel and θ is the angle whose tangent is the infill height to length aspect ratio.

The equivalent strut width of the panel is given by the expression provided by Mainstone (1971) as

$$\alpha = 0.175D^{(}\lambda_1 H)^{0.4} \tag{10}$$

3.1 Axial Hinge Property for Equivalent Struts

The axial hinge property for the equivalent struts are assigned manually in SAP2000. The capacity of the strut (R_{strut}) is determined based upon the masonry prism compressive strength and its shear strength.

$$R_{strut} = min(R_{cr}, \frac{R_{shear}}{Cos\theta_{strut}}) \tag{11}$$

Where, R_{cr} is the compressive strength of the masonry prism and R_{shear} is the shear strength of the masonry prism

3.1.1 Stress-Strain Relationship of Masonry

The model presented by Kaushik et al.(2007) is considered to simulate the compressive behavior of the masonry panel in this study. The model suggests that masonry stress- strain curve in compression can be considered as two parts, ascending parabolic part and descending linear part. The parabolic part can be represented by the following stress strain equation:

$$\frac{f_c}{f'_c} = 2\frac{\varepsilon}{\varepsilon'_c} - (\frac{\varepsilon}{\varepsilon'_c})^2 \tag{12}$$

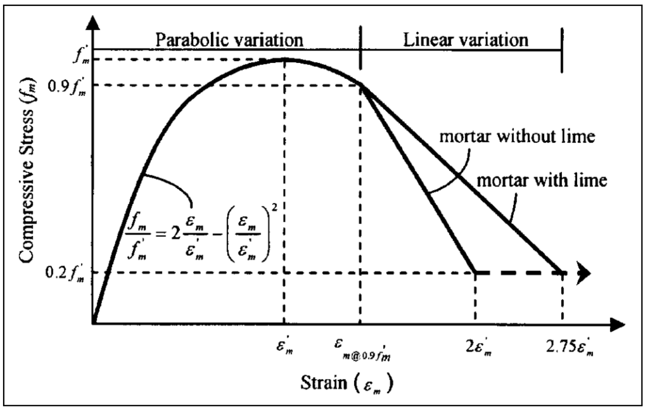

Figure 2: Idealized Stress- Strain relationship for masonry Kaushik et al(2007)

4. Model Description

For this study, a 2-dimensional model having 1,2,3 and 4 storey and 1,2,3 and 4 bays configuration with and without an equivalent diagonal strut is considered. The equivalent diagonal strut represents the effect of infill in the frame. The structure is analyzed and designed in compliance to IS800: 2007 for steel frame design and NBC 105:2019 for seismic resistant design. Models are studied for comparing the overstrength factor and ductility factor of steel frame structure with and without infill.

Table 1: Structural Details

Seismic Zoning Factor	0.4
Importance Factor	1
Soil Type	Medium Soil
Grade of Steel	Fe250
Storey Height	3m
Bay Length	5m
Infill Wall Thickness	230mm
Damping in Structure	5 %
Seismic Load	NBC 105: 2019
Frame Design	Seismic Coefficient Method

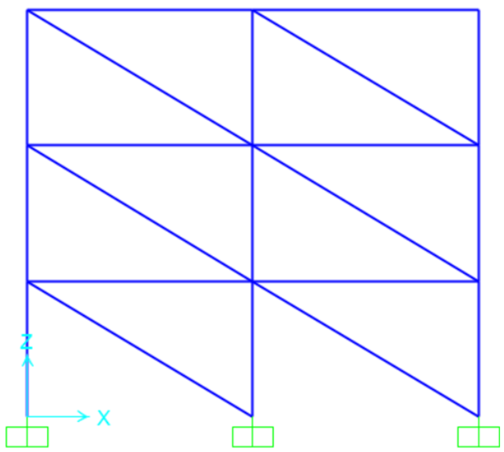

Figure 3: 2 Bay 3 Storey Infilled Model

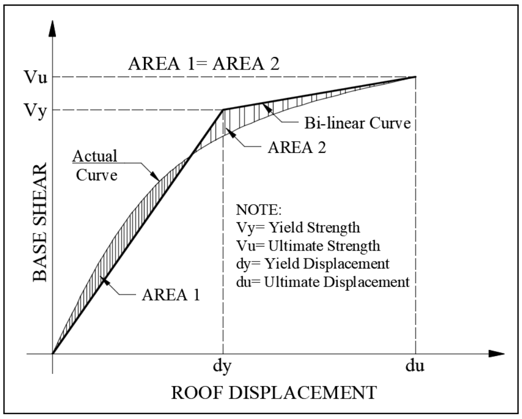

Figure 4: Pushover Curve and Idealized Bilinear Curve

The bare frame was designed to obtain the member sizes. Square hollow section (SHS) was used in Columns while I-Section was used in the beams.

Table 2: Loading on the Model

Slab Load	3.125kN/m^2
Imposed Dead Load	2kN/m^2
Typical Live Load	4kN/m^2
Roof Live Load	1.5kN/m^2
Total Dead Load(UDL)	(5.125)x6m=30.75kN/m
Typical Live Load(UDL)	4x6m=24kN/m
Roof Live Load(UDL)	1.5x6m=9kN/m
Wall Load(UDL)	12.265kN/m

5. Pushover Analysis

In this study, static nonlinear pushover analysis of steel frame models and masonry infill models were carried out in SAP2000. The pushover analysis of structure is carried out under dead loads and 30% live load and gradually increasing lateral loads. Geometric non-linearity was considered in the form of P-Delta effect in the analysis. Material non-linearity of frame element was represented by hinges based upon the force deformation criteria as per ASCE 41-13. In this study, discrete moment hinges(M) were assigned to all beams at its ends and P-M-M hinge were assigned to column at its two ends. Axial hinge (AH) was assigned to the equivalent strut at the center of strut. The structure was loaded laterally until the target displacement is reached.

The pushover curve obtained from the analysis is developed into a bilinear curve based on the provisions provided in FEMA 356: 2000. It states two conditions which include that i) the bilinear curve must have the same area below it as the pushover curve and ii) the first segment of the bilinear curve intersects the original curve at 60% of the significant yield strength. However, it was observed that multiple bilinear curves could be drawn in some cases which satisfied the above conditions and a third condition is required to obtain a distinct value of yield strength and yield displacement. In this study, visual inspection was carried out for all bilinear curves and the one which preserved the path followed by the original curve is used.

6. Results and Discussion

In this section, the analysis results of the model with and without the effect of infill have been discussed.

6.1 Effect on Overstrength Factor

6.1.1 Due to Number of Storey and Bays

In both the case i.e. with and without infill, the overstrength factor decreases with the increase in number of storey. This decrease of overstrength factor provides the indication that as the number of storey is increased, the design base shear is increased at a higher rate than the yield strength of the structure.

Also for a given number of storey, the number of bays did not seem to effect the overstrength factor by much. However for one storey model, the overstrength factor seem to decrease as the bay numbers are increased.

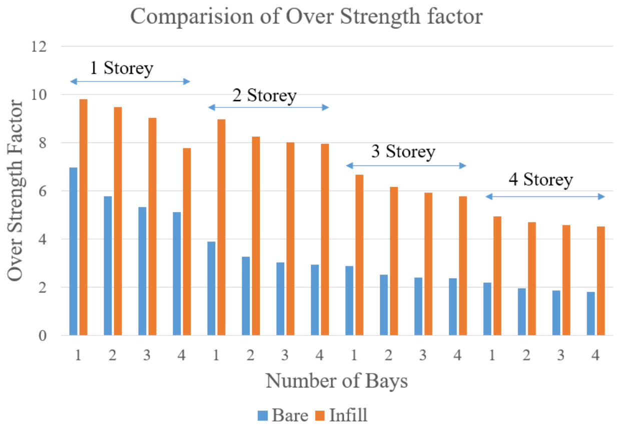

Figure 5: Comparision of Over Strength Factor

6.1.2 Due to Infill

Overstrength factor was comparatively higher for infilled frames. Its value ranged from 1.79 to 6.97 for bare frames while for infilled frame it ranged from 4.51 to 9.79. It can be seen that infills in frames increased the overstrength factor by at least 1.4 times than that for the bare frames.

6.2 Effect on Ductility Factor

6.2.1 Due to Number of Storey and Bays

It can be observed form figure 4 that in both bare frame model and the infilled model, that the ductility factor increased as the number of storey was increased. The effect of bays again seem to have very less effect on the ductility factor.

6.2.2 Due to Infill

The ductility factor is more for bare frames and for bare frame its value ranged from 1.46 to 3.72 while for infilled frame it ranged from 1.15 to 2.10. The ductility factor was decreasd by at least 0.52 times that for the bare frame.

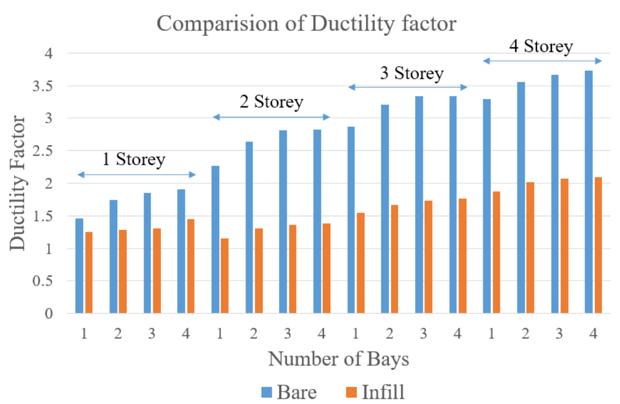

Figure 6: Comparision of Ductility Factor

In addition to this, the ability of equivalent strut to model the non linear behavior of masonry infill in SAP2000 was verified for a 2 bay 1 storey model by simplified micro modelling approach in ABAQUS.

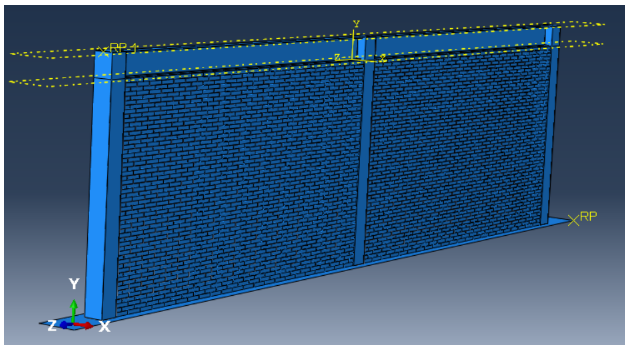

Figure 7: Masonry Infilled Steel Frame Model in ABAQUS

All the material propertied of masonry and infill used were same as used in SAP2000 except the modelling of the interface in which the normal stiffness (Kn) and shear stiffness (Ks) were used as per the recommendation provided by Lourenco, 1996. The peak tensile bond strength and peak shear bond strength were taken as per the recommendation of Nazir and Dhanasekar 2013.

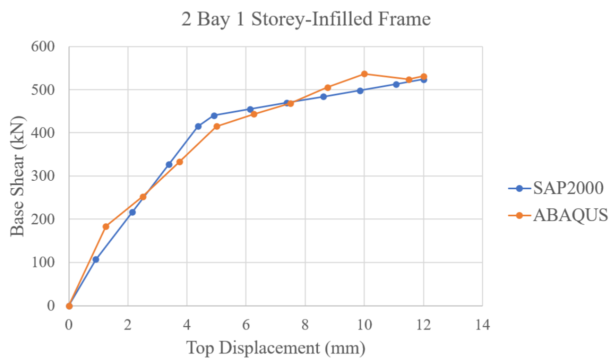

Figure 8: Comparision of Pushover Curves from SAP2000 and ABAQUS

From the figure 8, it was observed that both the pushover curves in SAP2000 and ABAQUS traced almost the same path. The macromodel developed in SAP2000 showed a value of yield displacement about 2.6% less than the mirco model and the yield strength form SAP2000 were observed to be 1.89% less than the micro models. It shows that the macro modelling approach used in SAP2000 using the equivalant strut model captured the behavior of infilled frame appropriately.

7. Conclusions

From the study of frame models with and without the infill using non linear static analysis, the following conclusions were drawn:

- The overstrength and ductility factor depends upon a lot of factors including the building geometry and infill. Using a single value of overstrength and ductility factor will result into inconsistent damage of buildings.
- The presence of infill in the frame increased the overstrength factor. The overstrength factor for both the bare frame and infilled frame decreased with the increase in the number of storey.
- The presence of infill in the frame decreased the ductility factor. The ductility factor for both the bare frame and infilled frame increased with the increase in the number of storey.
- The overstrength factor as well as ductility factor was found to be less dependent on the number of bays for infilled frames
- The two provisions available in FEMA 356: 2000 to develop a bilinear curve can be satisfied by multiple bilinear curves in some cases. Therefore a third condition is required to obtain a distinct bilinear curve.

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