LATERAL DUCTILITY AND STRENGTH OF COLUMN-FLAT SLAB SYSTEMS INCORPORATING THE ROLE OF MASONRY INFILL WALLS

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ABSTRACT :
Several advantages of the reinforced concrete flat slab systems over the classical moment resisting frames exist. The former offers larger unobstructed space, easier construction formwork, lesser construction duration and lower building height in addition to greater architectural flexibility. However, flat slab system suffers from some serious drawbacks: the absence of deep beams and shear wall results in low transverse stiffness which causes excessive deformations under the action of even moderate earthquakes, and the brittle slab punching failure due to transfers of shear and unbalanced moment between slabs and columns. In regions with high seismic hazard, flat slab systems are not recommended unless a sufficient lateral support system can be provided. Non-structural elements previously unaccounted for as part of the lateral support such as masonry infill panels may improve the seismic performance of flat slab systems. In this study, a method is proposed to predict the strength and ductility of a column-flat slab connection incorporating the action of masonry infill walls. This is followed by a parametric study on important design variables that govern the simultaneous plastic collapse behavior at the peak load. It is shown that carefully designed masonry infill walls allow for improved seismic resistant of column-flat slab systems.

KEYWORDS:
Ductile, flat, frame, infill, masonry, slab.

1. INTRODUCTION

It is recognized that the reinforced concrete flat slab system without any structural wall is considered less adequate to resist earthquake loading. However, flat slab structure is offers larger unobstructed space, easier construction formwork, lesser construction duration and lower building height in addition to greater architectural flexibility.

Designer's lack of knowledge regarding the seismic performance of the flat slab structure makes the decision to build a flat slab building difficult and hard. However, due to its potential architectural flexibility seismically active regions such as Indonesia still considers building flat slab structures in its big cities. The building story is kept below ten in order to keep the potential seismic-induced damage low.

In recent decades, researchers have studied how the flat slab system resisted lateral or earthquake type loading. A flat slab structure, in a way, behaves like the frame structure which resists the lateral loading by forming the plastic hinges at beam to column regions. Unlike, the behavior of the frame structure in such regions which is rather well understood, the slab-column interaction in resisting lateral loading in the flat slab structure is rather complex. It involves transfer of moment by punching shear in addition to flexural failure of the slab. Fortunately, recent studies (Luo et. al., 1995; Park and Choi, 2006; Park and Choi, 2007) reported that such a complex behavior is possible to model using the equivalent frame method.

In this work, the equivalent frame method is used to approximate the flat slab behavior in resisting earthquake loading. The approach brought forward by Luo et. al., 1995 is closely followed. The method is used to generate a pushover response of a typical 10-story building. Then, the effect of incorporating the contribution of masonry infill toward the lateral response of the same building is studied. Non-structural component such as an infill wall has been shown to contribute significantly to the pushover response of a building (Saneinejad and Hobbs, 1996).

2. MODELING FLAT SLAB BEHAVIOR

The approach taken to model the flat slab follows closely that by Luo et. al., 1995. In this study a nonlinear pushover analysis is also performed to investigate the ductility of the structure. Briefly, the approach consists of finding the effective width of a strip of the flat slab structure. The plan view of a flat slab strip can be seen in Fig. 1. The following equations have been used to determine the effective width:

$$\chi = 1 - \frac{0.1V_g}{A_c\sqrt{f_c'}} \tag{2.1}$$

$$K_t = \frac{9E_{cs}C}{l_2\left(1 - c_2/l_2\right)^3}, \quad K_s = \frac{4E_{cs}I}{l_1} \tag{2.2}$$

$$\alpha_e = \frac{K_t}{K_t + K_s} \tag{2.3}$$

where E_{cs} is the Young's modulus of concrete, V_g is the shear force at the slab-column joint due to gravity load, and I is the moment inertia of the slab.

The equivalent frame uses concrete with f_c'= 30 MPa and steel rebar with f_y=413 MPa. The Young's modulus of concrete is 29 GPa and that of steel rebar is 200 GPa. The equivalent frame shown Fig. 2, is subjected to several combinations of dead load, live load, and earthquake load with load factor given in Table 2.1. The loading magnitude is obtained from Indonesian standard (SNI, 2003). Briefly, the building is to be designed using the static equivalent loading for an earthquake zone 3 according to the Indonesian earthquake zone mapping. Jakarta, the country's capital city belongs to this zone. The analysis is performed with the structural analysis software SAP 2000. The software produces the maximum moments and shears in the members. The sizes and amount of steel reinforcement of the structural members are designed based on this result. A strong column weak beam/slab criterion is adopted.

A measure of ductility of the structure is defined as the ratio between the displacement and the yield displacement. Considering the flat slab system as an ordinary moment resisting frame, it is designed to withstand a value of ductility equals 2.1. This corresponds to the earthquake load reduction factor R of 3.5. The designed members are named such as shown in Fig. 2. There are only three types of members: K1 for columns of 1st and 2nd floors (500 mm by 500 mm with 8ϕ25), K2 for columns of 3rd-10th floors (500 mm by 500 mm with 8ϕ20), and 200 mm thick slab. The slab reinforcement is assumed to be governed by the maximum ultimate moment at the slab-column joint M_u. The flexural capacity M_f and the shear capacity V_v of the slab is determined as follows

$$M_f = \frac{\gamma_f}{\phi}M_u \tag{2.4}$$

$$M_V = \frac{\left(1-\gamma_f\right)}{\phi}M_u \tag{2.5}$$

$$M_V = \left[v_u\left(c_2 + d\right)d\right]\left(c_1 + d\right) = V_v\left(c_1 + d\right) \tag{2.6}$$

$$M_f = \frac{\gamma_f}{\phi}M_u \tag{2.7}$$

where γ_f is the coefficient for the flexural contribution towards the joint capacity and ϕ is the strength reduction factor. The other notations can be found from Fig. 1.

The slab flexural and shear capacities determined from above are then used as inputs in the pushover analysis. The analysis procedure follows that provided by SAP 2000. In this procedure, a member on which plastic hinges can form is assigned the constitutive relation such as shown in Fig. 3. This relation is not only for moment-rotation but also for shear force-displacement and axial load-displacement. In such relations, the maximum moment or shear in the slab are assigned values obtained using M_f and V_v in (2.4-2.6). The behavior will be dependent on what occurs first, shear or flexural failure. While M_f and V_v can be found, the other model parameters such as the rotation/displacement at peak load have been estimated from data in (Park and Choi, 2006; Park and Choi 2007). The reinforcement detail needed to achieve M_f and M_v can be found using the method suggested in (Park and Choi, 2006; Park and Choi 2007).

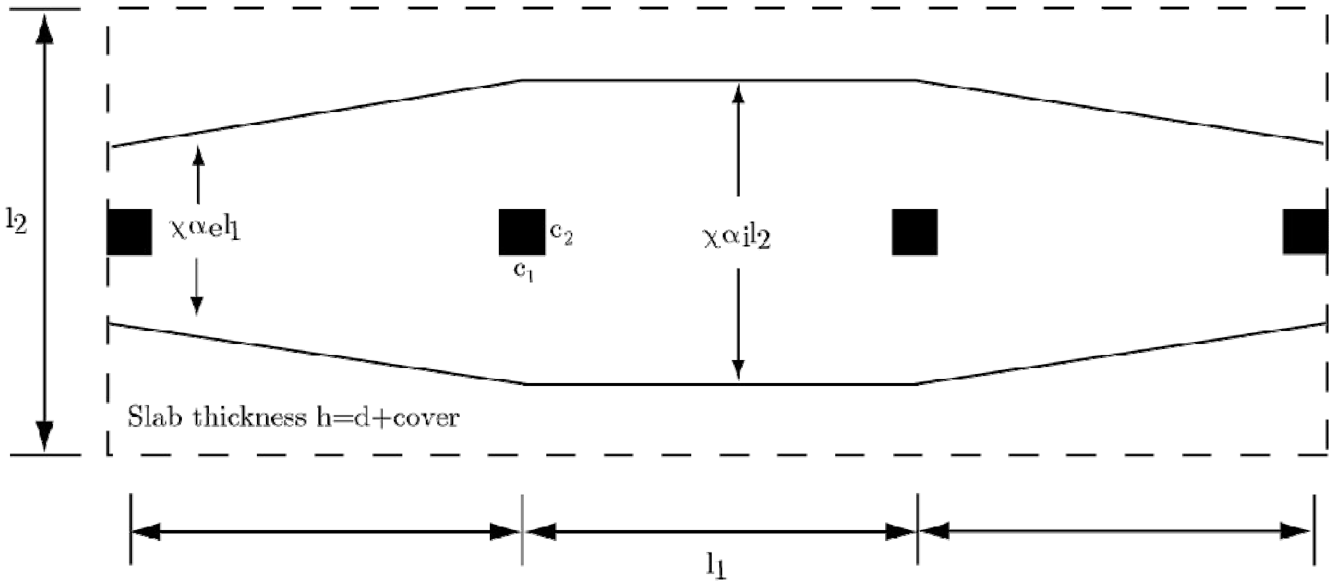

Figure 1 Equivalent width of flat slab strip

Table 2.1 Load combinations

Combination	Dead load factor	Live load factor	Earthquake load factor
1	1.4	-	
2	1.2	1.6	
3	1.2	1	1
4	1.2	1	-1
5	0.9		1

The study continues to the frame behavior incorporating the infill wall. The modeling is achieved by replacing the wall with a strut. The behavior of the strut is defined through the load-displacement behavior similar to Fig. 3. Three infill wall/strut configurations have been considered (Fig. 4). The data regarding the effective dimensions, strength, and stiffness of the infill wall have been adopted from (Saneinejad and Hobbs, 1995). Three infill types have been considered: infill 1 is 600 mm by 100 mm strut with average compressive strength of 375 kN, infill 2 is 800 mm by 100 mm strut with average compressive strength of 371 kN, and infill 3 is 600 mm by 150 mm strut with average compressive strength of 349 kN.

3. RESULTS

First, let us consider the effect of infill configuration on the left of Fig. 5. The plot shows the base shear, which is the reaction shear force over the base columns, and the roof ductility which has been obtained by dividing the roof displacement by the displacement at yield. The assessment is such that the frame response that gives as large ductility as possible at the peak load is considered a good structure. Keep in mind that in the preceding section the frame was targeted to withstand a ductility at peak (or peak ductility) as much as 2.1. The bare frame appears to be quite ductile as the base shear change very little over a ductility range of around 10. However, it gives a moderate base shear capacity. Infill configuration 1 does not increase significantly the capacity and provides a peak ductility of 2.5. Infill configuration 2 increases the capacity to almost three times the capacity of the bare frame and gives a peak ductility of 4.3. significantly. Infill configuration 3 increases the capacity to almost twice the capacity of the bare frame but gives only a peak ductility of 2.1.

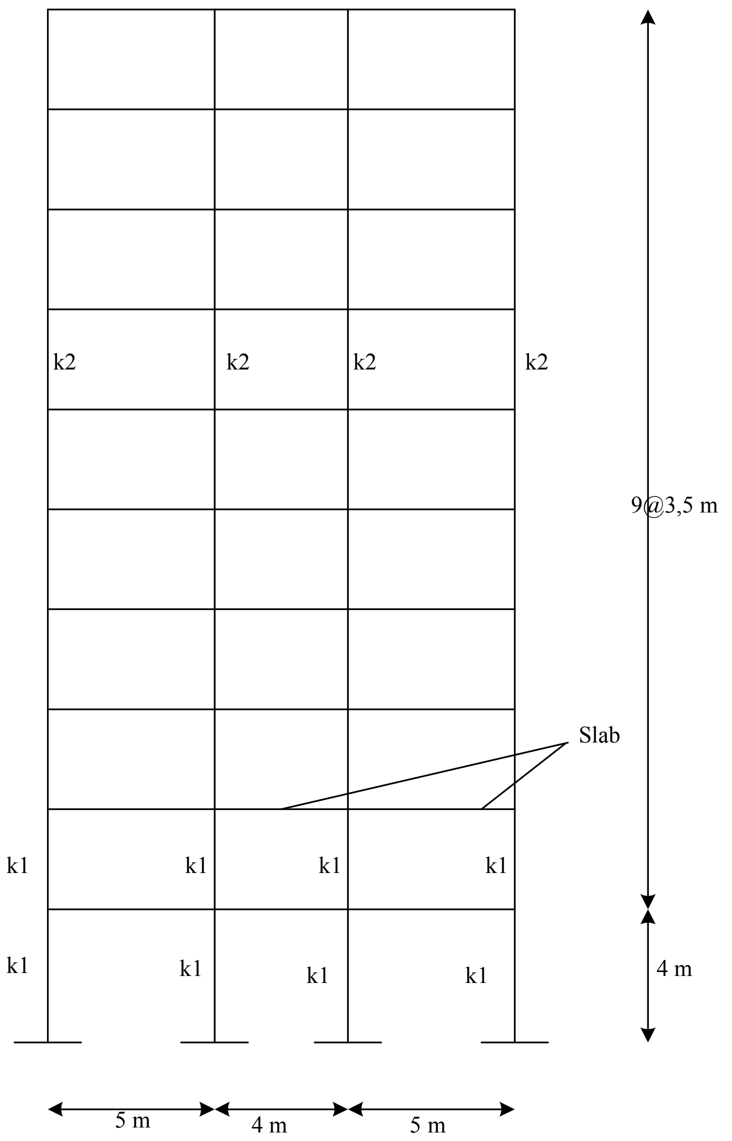

Figure 2 10-story building prototype with types of columns and slab

The effect of different infill properties is shown on the left of Fig. 5. It is seen that the effect is minor. This is to be expected since the variation of masonry strength in each infill type is not siginificant. It is also worth mentioning that the properties of masonry normally do not vary significantly from one place to the other. So the effect of infill upon the structural behavior is likely to be governed by the configurations.

The deformed state and the plastic hinges configurations of different infill types can be seen in Fig. 6. A closer look upon the deformation of infill configuration 3 indicates a soft story mechanism taking place. This can be seen from the large lateral drift at 2nd and 3rd floors compared to drift at any other floors. The mechanism also allows the formation of plastic hinges at the middle part of the frame where infill is not present. This is the effect of infill in stiffening the exterior part of the frame while at the same time allowing the columns in the interior/middle part to largely deform.

It is worth mentioning that, shear failure is allowed in the analysis. This is necessary for the flat slab as it is normally accompanied by the punching shear when fails. In defining the shear behavior similar to Fig. 3, the shear deformation up to peak load is set quite low thus producing brittle behavior.

Fig. 6 also shows that the unfilled frame configuration 3 does not result in excessive deformation of base columns. Furthermore, the plastic energy dissipation occurs more uniformly in most members. The low peak ductility is likely caused by the stiffening effect of the strut. This affects the distribution of moment and shear in the slab/columns. Since the less ductile shear failure is allowed in the slab the brittle effect of shear is seen more dominant on the overall response.

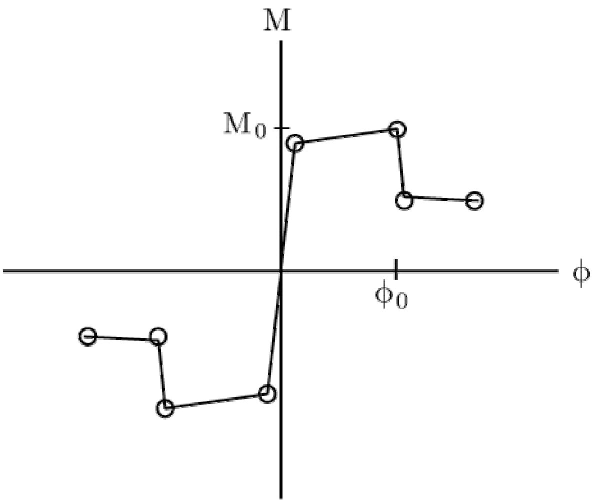

Figure 3 Moment/load versus rotation/displacement

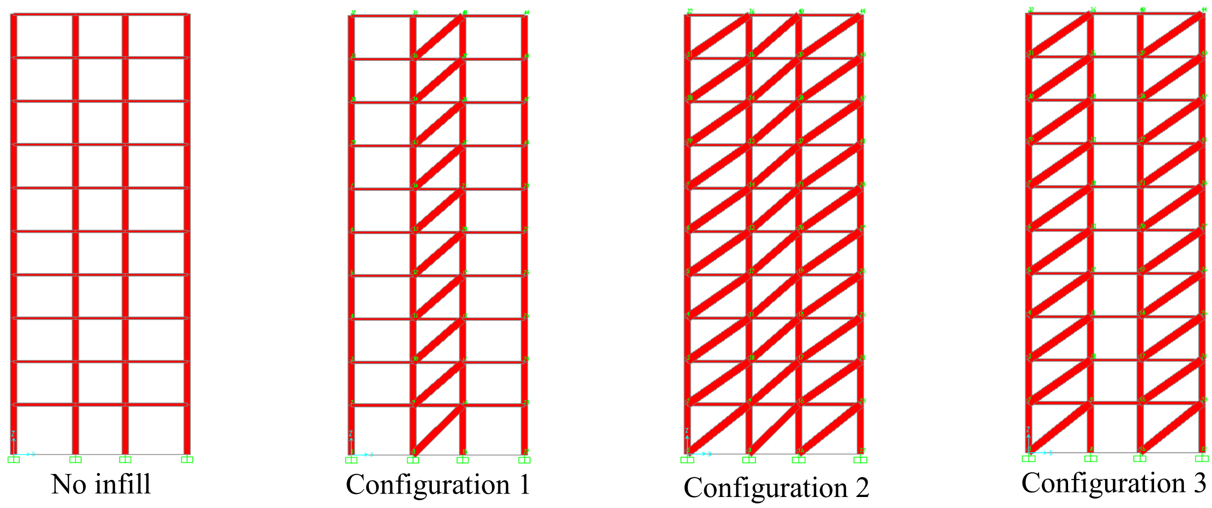

Figure 4 Unbraced frame and configuration types of strut supported frames

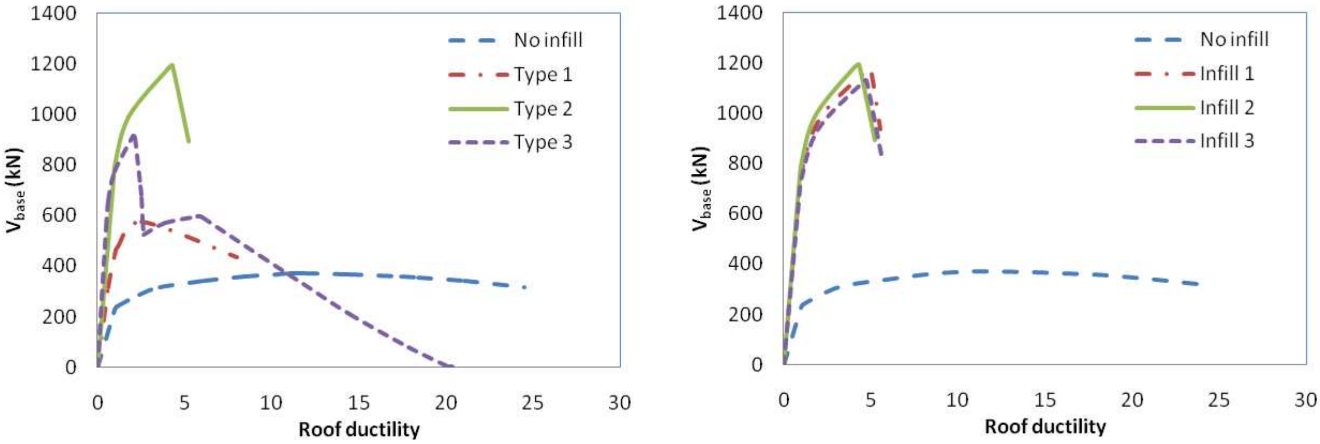

Figure 5 Base shear of the frame against roof ductility.

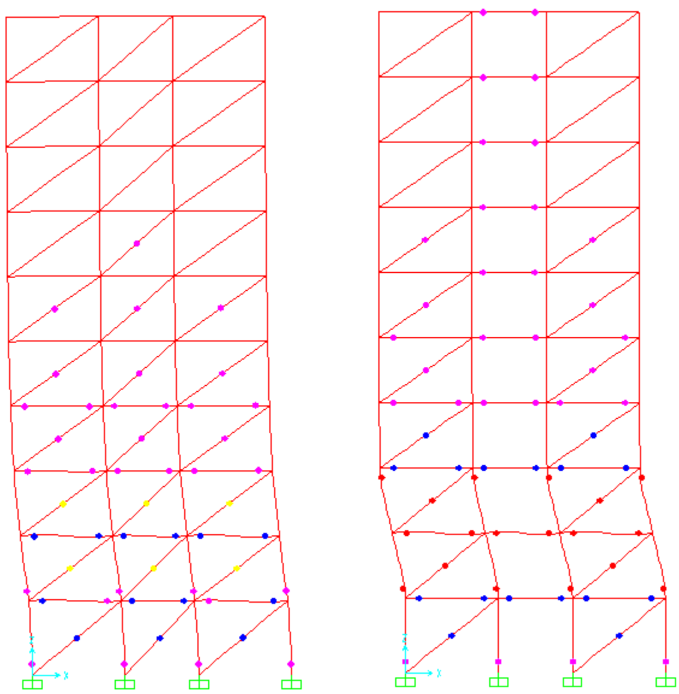

Figure 6 Deformed states and plastic hinges locations in different infill types

4. CONCLUSIONS

The flat slab structure is normally chosen in a seismically active region. However, it allows for greater architectural flexibility and may be chosen if it can be designed accordingly. The method presented herein uses the equivalent frame concept to design the structural members of the flat slab system. Then the ductility is assessed using the pushover analysis. This allows for a safer yet efficient design where the prescribed ductility can be confirmed.

Masonry infill walls are normally present in a building but unaccounted for in the design calculation. The method presented herein allows for analyzing the contribution of masonry infill walls. The result shows that different infill configurations significantly alter the pushover response of the structure. It is found that base shear capacity of the unfilled frame can be increased up to three times that of the bare frame. However, the peak ductility of the unfilled frame is significantly lower. The explanation is because the less ductile shear failure is allowed in the slab. In filled frames, the infill tends to stiffen the frame and increase the load to the slab and column. The shear failure occurs as a result of this thus, producing a less ductile pushover curve.

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