

Seismic Evaluation of a Low-Rise Reinforced Concrete Commercial Building by the Capacity Spectrum Method

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Abstract: This paper presents seismic evaluation of a typical low-rise reinforced concrete commercial building based on the capacity spectrum method (CSM) recommended in ATC-40. A nonlinear three-dimensional finite element (FE) model is developed for assessing the seismic capacity of the structure. To simulate a more realistic behavior of the building during ground motions, effects of foundations, masonry infills, and other site-specific features are integrated into the model. Seismic capacity of the structure is determined using pushover analysis and compared with the demand imposed by predicted ground motions. Results indicate that this building possess the most undesirable brittle failure with low level of seismic capacity. Further, this analysis method reveals several earthquake vulnerable features and the effectiveness of 3D modeling of buildings with strength and stiffness irregularities. Finally, design recommendations for eliminating earthquake vulnerable features are proposed. However, the typical details used at the foundation-superstructure connection limits the structural system seismic capacity.

Keywords: Capacity Spectrum Method, Building, Earthquake Vulnerability, Pushover Analysis, Soft-Story.

1.0 Introduction

Typical low-rise commercial buildings have the first floor taller than the other floors or have relatively less number of infill panels and partition walls. Seismic performances of these buildings are pretty complex. The most common failure mode of these buildings is the weak and/or soft story mechanism. A weak storey is defined as the one in which the storey strength is less than 80 percent of that of the storey above. A soft storey is defined as the one in which the lateral stiffness is less than 70 percent of that in the storey immediately above or less than 80 percent of the average stiffness of the three stories above.

During past earthquakes, many of them collapsed while others survived. Around 280 earthquakes have occurred from 1988 to 1992, about 59,940 persons were killed, and 1769,000 persons became homeless [0]. During the 1990 Iran earthquake, more than 130,000 houses and commercial buildings were damaged. More than 105,000 houses and 320 apartment buildings were damaged by the 1989 Loma Prieta earthquake. Most of the buildings that failed in Turkey, 1992, were modern buildings designed and particularly, constructed without compliance with seismic code regulations. Ruiz et al., [0] has considered the seismic

performance of buildings with weak first story during the Mexico City earthquake in 1985. The idea of a soft first story had been formerly proposed as a means to reduce the seismic response of buildings because of its action as a base isolator. But, 8% of the damaged buildings in the Mexico City area had a weak first story. In the same time Miranda and Bertero [0] also have considered the same earthquake and have mentioned that 39% of engineered buildings that collapsed or suffered severe damage were buildings of five or fewer stories. The percentage of this type of building that collapsed or suffered severe damage was significantly smaller than the percentage of 7-to 15-story buildings that were severely damaged. Overstrength is the primary reason for the survival of most of these low-rise buildings. The number of totally collapsed buildings during the 1999 Chi-Chi earthquake was 20,815. The reasons for such serious building damage were due to very high-intensity ground motions, bad construction quality and improper structural system [0]. Among those collapsed and severely damaged buildings, street-front buildings, school buildings, and town halls were dominant. A majority of the damaged buildings had shear walls in only one direction. So, the buildings were very strong on wall direction while their capacities could be just equal to the capacity specified by the code in the direction

without wall [0]. In India, reinforced concrete frame buildings with brick masonry infills have shown excellent performances, during past two moderate earthquakes, even though most such buildings were not designed and detailed for seismic response [0]. Further, they have mentioned that the survival of aforementioned buildings was mainly due to the beneficial influence of masonry infill walls. Asymmetric structures, in terms of mass or stiffness distribution, suffer an additional response due to the coupling of lateral and torsional motions. This coupling motion can induce higher demands on resisting elements than lateral motion alone and, unless properly accounted for in the structural design, may result in severe damage or collapse [0].

One of the most effective ways of minimizing potential earthquake related losses is to conduct reliable assessments of the vulnerability of existing structures and to develop and implement effective ways to upgrade structures identified as hazardous. Further, the lessons from such assessments can be used to revise analysis, design, and detailing guidelines. Both elastic (linear) and inelastic (nonlinear) methods are available for analysis of existing structures. They are static lateral force procedure, demand/capacity ratio method, secant method, non-linear time history analysis method, and non-linear static pushover analysis method. Elastic analysis methods cannot predict a realistic behavior of a structure. Therefore, inelastic analysis methods are used to study behavior of structures under earthquake loading. The most direct or realistic inelastic analysis method is the complete time history analysis, which is considered overly complex and impractical for general use. Most promising and simplified analysis is the nonlinear static pushover method. In fact, the pushover analyses can give information on the structural strength capacities and the deformation demands. It can also expose design weaknesses that may remain hidden in an elastic analysis. Such weaknesses include story mechanisms, excessive deformation demands, strength irregularities, and overloads on potentially brittle elements, such as columns and connections.

1.1 Capacity Spectrum Method (CSM)

The CSM had been introduced in several guidelines for seismic evaluations such as the ATC-40 [0] and FEMA-273 [0]. This is an approximate procedure to analyse the seismic response of a structure with a nonlinear static analysis (pushover). By applying nonlinear static pushover method, it is possible to compute the relation between the total lateral force on a structure against the lateral deflection of the

roof of the structure. This curve is often referred to as the 'pushover' or capacity curve, which represents the overall behavior of the structure. The transformed capacity curve from shear force versus roof displacement coordinates into spectral acceleration versus spectral displacement coordinates, Acceleration-Displacement Response Spectra (ADRS), the *capacity spectrum* is developed. Performance evaluation of a structure is required to have the response spectrum (or demand spectrum). Demand is the minimum required deformation and strength capacity levels of a structure required to withstand during a considered earthquake ground motion. The plot of estimated displacements in ADRS format is called the *demand spectrum*. The capacity spectrum method, a nonlinear static procedure which provides a graphical representation of the global force-displacement capacity curve of the structure and compare it to the response spectra representations of the earthquake demands, is a very useful tool in the evaluation of existing structures. Results from many applications of this method confirm its robustness and accuracy as a reliable analysis method for evaluating existing structures [0].

2.0 Objective and Scope

The main objective is to examine the seismic performance of a typical, low-rise, reinforced concrete, commercial building by using the *Capacity Spectrum* method. The results is used to propose retrofitting methods and some guidelines, to be considered in designing such buildings, to avoid possible earthquake hazards during future earthquakes.

To achieve the aforementioned objectives, a typical, low-rise, reinforced concrete commercial building is selected and the ATC-40 [0] recommended seismic evaluation procedures are employed.

3.0 Modelling for Evaluation

3.1 Flexural Behavior

In this analysis, it is required to find yield moments of all the flexural members. There is no specific formula suggested in ATC-40 [0] to calculate the yield capacity of concrete members. Hence, a well-accepted and commonly used method documented in literature [0, 0] is used.

3.2 Shear Behavior

Shear is rather complex to handle because so-called "shear failure" is a failure under combined shearing force and bending moments, sometimes, axial load, or torsion, or both may act simultaneously. Design

code equations for shear capacity are conservative when compared with test results. The expected component strength needs to be used for structural evaluation. Further, the analysis methodology used for structural evaluation is displacement based. According to ATC-40 [0], the expected strength is defined as the mean maximum resistance expected over the range of deformations to which the component is likely to be subjected. Hence, the equations proposed in the following sections are used in this analysis.

3.2.1 Shear Strength of Columns

Reinforced concrete columns are subjected to axial compression or axial tensile and shear forces due to combined effect of gravity and lateral loads. Axial compression tends to increase the shear capacity while axial tension decreases it.

3.2.1.1 Concrete Shear Capacity (V_c)

ACI design code equations [0] for concrete shear strength of columns under axial compression and axial tension are compared with experimental results as shown in Figure 1. Both the equations provide a safe lower bound to the strength. Eq. 1, proposed by ATC-40 [0], represents average value of test data for columns under axial compression ($k=1$ and $\lambda=1$ are used).

$$V_c = 3.5\lambda \left(k + \frac{N}{2000 A_g} \right) \sqrt{f'_c} b_w d \dots\dots (lb) \quad (1a)$$

$$V_c = 0.29\lambda \left(k + \frac{N}{2000 A_g} \right) \sqrt{f'_c} b_w d \dots\dots (N) \quad (1b)$$

Where $k=1$ in regions of low ductility and 0 in regions of moderate and high ductility, $\lambda=0.75$ for lightweight aggregate concrete and 1 for normal-weight aggregate concrete, N = axial compression force (zero for tension force), and A_g = gross cross sectional area.

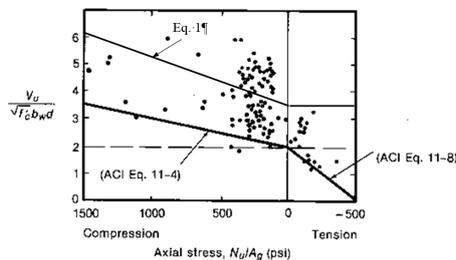


Figure 1: Effects of axial loads on concrete shear strength (Experimental data source [0])

According to Eq. 1a and its conditions, under tensile forces in regions of low ductility, concrete possess constant shear strength and shows contradictory behavior with the test results presented in Figure 1. Priestley et al. [0], presents a model for concrete shear strength without axial force effects.

According to that model concrete possesses constant shear capacity under low ductility. Columns in typical low-rise buildings are seldom subjected to tensile forces. Considering the above facts, it is recommended to use Eq. 2 for columns with low ductility or small tensile forces.

$$V_c = k \sqrt{f'_c} A_e \quad (2)$$

where, effective shear area, $A_e=0.8A_g$.

3.2.1.2 Shear Capacity of Stirrups (V_s)

The equation recommend by ATC-40 [0] is given below.

$$V_s = \frac{A_v f_y d}{0.6s} \quad (3)$$

Where A_v is the area of the shear reinforcements, s is the spacing of stirrups, and f_y is the yield strength of the shear reinforcements. Eq. 3 is derived from Eq. 4 by assuming that the angle between the compression diagonals and the column axis as 30° .

$$V_s = \frac{A_v f_y d}{s} \cot \theta \quad (4)$$

In the presence of axial compression, the diagonal cracks tend to be flatter than 45° [0]. Priestley et al. [0] proposed an equation similar to Eq.3 with $\theta = 30^\circ$ and confirmed its validity after comparing its predictions with experimental data.

Tests have consistently demonstrated that the angle of inclination of the diagonal cracks is not noticeably affected by axial tension and that the shear resisting mechanism of truss action remains operative [0]. Hence, Eq. 5 is suitable for calculating shear capacity of transverse reinforcements of columns under axial tension.

$$V_s = \frac{A_v f_y d}{s} \quad (5)$$

3.2.2 Shear Strength of Beams

3.2.2.1 Concrete Shear Capacity (V_c)

There is no particular formula suggested in ATC-40 [0] to calculate concrete shear strength of reinforced concrete beams. Eq. 6 is in given in ACI 318 [0] to calculate V_c .

$$V_c = \left(1.9\sqrt{f'_c} + 2500 \frac{\rho_w V_u d}{M_u} \right) b_w d \leq 3.5\sqrt{f'_c} b_w d \dots (lb) \quad (6a)$$

$$V_c = \frac{1}{6} \left(\sqrt{f'_c} + 100 \frac{\rho_w V_u d}{M_u} \right) b_w d \leq 0.29\sqrt{f'_c} b_w d \dots (N) \quad (6b)$$

Where $\rho_w = A_s / b_w d$ is the reinforcement ratio, V_u is the factored shear force, and M_u is the factored

moment. The application of Eq. 6a for continuous beams has been subjected to disputation. The shear span (a) is equal to M_u/V_u , which shows that there is a support to accommodate a compression strut at zero moment position. But for continuous beams there is no support to carry a compression strut reaction at the zero moment point. Hence, Eq. 7 is recommended for continuous beams [0].

$$2\sqrt{f'_c} b_w d \dots\dots\dots(lb) \quad (7a)$$

$$0.166\sqrt{f'_c} b_w d \dots\dots\dots(N) \quad (7b)$$

Eq. 7 does not incorporate the effects of longitudinal reinforcements on shear strength of concrete. Eq. 8 accounts the shear strength contribution from longitudinal reinforcements through the dowel action [0].

$$v_c = v_b = (0.85 + 120 \rho_w) \sqrt{f'_c} \leq 2.4 \sqrt{f'_c} \dots(psi) \quad (8a)$$

$$v_c = v_b = (0.07 + 10 \rho_w) \sqrt{f'_c} \leq 0.2 \sqrt{f'_c} \dots(MPa) \quad (8b)$$

Eq. 8 is compared with the test results. Within the practical range of reinforcement ratio, $\rho_s = A_s/b_w d$, it represents more realistic values while Eq. 7 represents the minimum values. Therefore, Eq. 8 is selected for this analysis.

3.2.2.2 Shear Capacity of Stirrups (V_s)

Generally, both concrete and the web reinforcements carry the shear acting on any reinforced concrete member. Shear strength (V_s) predicted by the well-known truss analogy is used in this analysis. Axial force acting on a beam is considered small; thus, the angle between longitudinal axis of the member and the diagonal compression struts is assumed 45° . Eq. 5 is used in this analysis to calculate shear strength contribution of web reinforcements that are perpendicular to the longitudinal axis of the member.

3.3 Development Length and Lap Splices

Ductility of structural components that are mainly designed for gravity loads is very low. One set of equations in ACI 318 [0] is for the component yielding regions with low ductility demands and outside the yielding regions. The other set of equations in ACI 318 [0] is for the detailed requirements and strength provisions of straight, hooked, and lap-spliced bars within the yielding regions of components with moderate and high ductility demands. Most of the recommendations given in Paulay and Priestley [0] are related to components with moderate or high ductility demands. Recommendations given in ACI 318 [0] and Paulay and Priestley [0] for components with moderate and high ductility demands were

compared and yielded more or less similar results. Thus, in this analysis, the ACI equations are used, and all the conditions given under each equation are adopted.

In case, if the development, hook, and lap-splice length and detailing are not in compliance with ACI 318 [0], Eq. 9 recommended in ATC-40 [0] is used to calculate the maximum stress capacity, (f_s), of reinforcements.

$$f_s = (l_b/l_d) \times f_y \quad (9)$$

Where, l_b is the length provided for development, hook, or lap splice; l_d is the length required by ACI 318 [0] for development, hook, or lap splice.

4.0 Modelling of Existing Structure

The selected building represents the most common type of typical low-rise reinforced concrete commercial buildings (Figure 2). It has five stories, including a mezzanine floor. The building is rectangular in plan with an overall dimension of approximately 12 m by 36 m in the north-south and east-west directions, respectively.



(a) Front view (facing south)

Figure 2: Front and side views of the building

Infill walls are provided only in the outside frames (Figure 2 and Figure 3). However, east-west direction have doors or windows with partial infills (Figure 3 e and f). Frame B and frame C have no beams in the east-west direction at the fourth floor and the mezzanine floor levels, respectively (Figure 3).

All the column sizes and reinforcement detailing are changed at the 3rd floor level. From foundation to the 3rd floor level column size is 0.25m×0.25m, and at the 3rd floor level it changes to 0.2m×0.2m causing abrupt changes in stiffness and strength of the structure. Height measured from the first floor to the mezzanine floor, between frames A and B, is 2.6m. The mezzanine floor level between frames B and D is 0.65m higher than that at frames A and B, Figure 3. All these features cause vertical and plan irregularities of the structure. It is obvious that this building is weaker in the east-west direction than the north-south direction. Thus, the analysis is carried out only in the longitudinal direction. Mainly, there are four frames in that direction.

They are frame A, B, C, and D that carry gravity and lateral loads during earthquakes. Hence, seismic capacity of the structure and behavior of those frames under the gravity and lateral load combination and the failure mechanisms were studied.

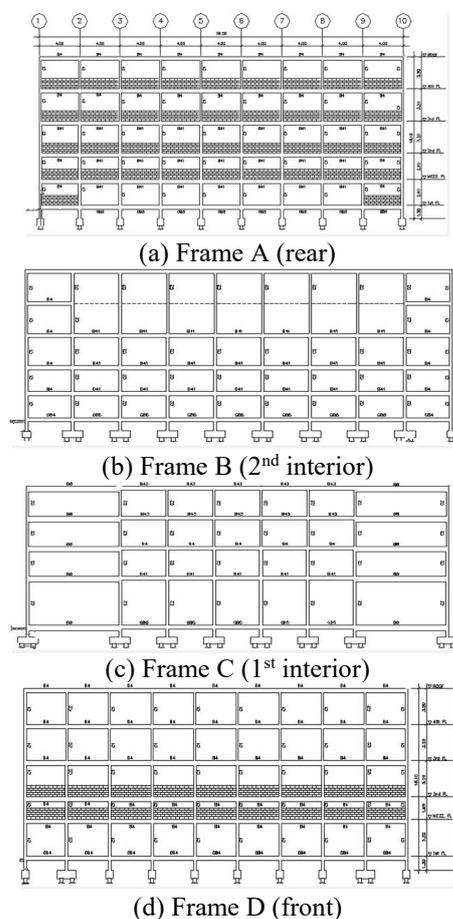


Figure 3: Longitudinal frames of the building

Modeling of structural components is carried out mainly by adopting the recommendations given in ATC-40 [0] and the equations presented in section 3. For modeling of non-structural component, masonry infill panels, FEMA-273 [0] guidelines are used with necessary modifications as presented in section 4.5.2.

4.1 Computer Models

For the analysis, several different nonlinear FEM models were developed using SAP 2000 [0], two-dimensional inelastic models for frames A, B, C, and D with fixed-base, three-dimensional inelastic models with fixed-base, flexible-base (with foundations), and flexible-base with masonry infill panels. These models were constructed using as-built drawings and the best models of individual components to simulate a more realistic behavior in the nonlinear range of the structure and to accommodate all the irregularities and the other unfavorable features in the existing building.

4.2 Loads

4.2.1 Gravity Loads

Structural response to an earthquake depends on the magnitude of gravity load present at the time it hits the building. For the most reliable evaluation of the structural response, the gravity load should consist of dead load plus most likely live loads. ATC-40 [0] presents the most likely live loads for various occupancies based on a survey conducted in Washington, D. C. In this analysis, direct use of these values is not realistic. Thus, by comparing the design live loads given in Uniform Building Code (UBC) with the values in ATC-40 [0], it is necessary to find the most likely live load as a percentage of the design live load.

For this analysis, dead and most likely live loads are considered. For the dead load, the density of concrete is assumed as 2400 kg/m^3 . Dead load of half and full width masonry walls are 90 kg/m^2 and 180 kg/m^2 , respectively. Most likely live loads are calculated as 45% and 30% of the design live loads for residence and storage areas, respectively. As per ATC-40 [0] guidelines service loads are used.

4.2.2 Lateral Loads

For low-rise buildings, the first mode of vibration is dominant and generally the shape is idealized as an inverted triangular. If plan or vertical irregularities exist, inverted triangular load pattern assumption may not yield accurate enough results. In that case, the loading pattern according to the first mode shape is required. In this analysis, both the inverted triangular and the first mode shape load patterns are considered. The loads are applied as concentrated loads at the floor levels (at beam-column joints).

To determine mode shapes, joint masses are required. In the mass calculation, contributions of all the structural and non-structural components are considered. Building masses are simplified as lumped masses at the beam-column joints.

4.3 Load Deformation Relationship

Nonlinear load-deformation relation is used in reinforced concrete component modelling (Figure 4). This load-deformation relationship is used to represent the moment-curvature, shear force-shear angle, and axial force-axial deformation relations of the components. During the analysis, gravity load is applied prior to the lateral load. Thus, lateral loading starts at a point other than the origin of the load-deformation relation. The slope from A to B of the Figure 4 corresponds to the fully cracked stiffness of flexure-dominated components and uncracked stiffness for shear-dominated

components. Point B represents the yield strength. The slope between B and C represents the strain hardening of reinforced concrete components. Point C corresponds to the ultimate strength. The sudden drop from point C to D represents the initial failure of the component. Residual strength of the component is indicated from point D to E. Total collapse that occurs by losing its gravity carrying capacity is denoted by the sudden drop at point E.

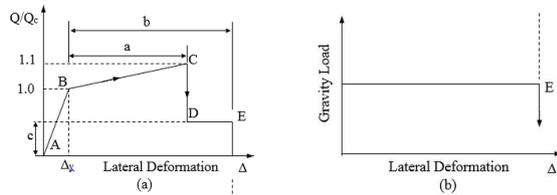


Figure 4: Generalized load-deformation relation for reinforced concrete components

4.4 Material Properties

For the evaluation of existing buildings, properties need to be determined by inspection and testing. Concrete modulus of elasticity is calculated using Eq. 10.

$$\dots E_c = 4700\sqrt{f'_c} \dots (MPa) \dots \quad (10)$$

Concrete compressive strength of 20.6 MPa for structural components and 35 MPa for piles is used. Steel used in construction sites shows larger variation between nominal yield strength and the actual yield strength [0]. Yield strength of 390 MPa and elasticity modulus of 200 GPa are used to calculate flexure and shear capacities of beams and columns. Yield strength of steel 1.62 GPa is used for piles. Compressive strength of 4 MPa and elasticity modulus of 1.275 GPa are used for masonry [0, 0].

4.5 Structural Modeling

Analytical model for the evaluation must represent all the influential components in complete three-dimensional characteristics of building behavior, including mass distribution, strength, stiffness, and deformability. The effective initial stiffness is calculated based on ATC-40 [0] recommendations.

4.5.1 Beam-Column Frames

The beam-column frame model should represent the strength, stiffness, and deformation capacity of beams, columns, and beam column joints. Beam-column joints are assumed rigid because they are monolithically cast.

4.5.1.1 Beam

Flexure, shear, and development length effects are the three main factors considered in modeling of beam components. In the analytical model of the beam, all the properties are concentrated at the

component centerline. This analyzed building has pre-cast slabs at each floor level except at the roof. The stiffness and strength contribution of precast slabs as well as the roof slab are not considered.

Beam flexural and shear capacities vary along the length of the component. Generally, it is required to provide plastic hinges along the centerline so that the beam can develop inelastic response. In this analysis, plastic hinges are assigned only at critical sections. Following the reinforcement detailing given in the drawing, hinges are provided at both ends, midspan, and at intermediate sections based on the reinforcement cut-off details.

4.5.1.2 Column

Two main influential factors considered are shear and flexure with the axial force interaction. In the analytical model of the column, all the properties are concentrated at the component centerline. It is required to define axial force-moment interaction yield surfaces for each assigned flexural plastic hinge. During the analysis, variation of axial force is not significant. Thus, the axial force due to gravity load is selected as the effective axial force for defining necessary parameters.

The analysis option “Restart Using Secant Stiffness”, which is recommended in ATC-40 [0], is selected for the pushover analysis. With this option the axial-moment (P-M-M) hinge available in SAP 2000 Nonlinear does not work properly. Therefore, the flexural hinge is assigned to the columns instead of the P-M-M hinge. The yield moment for the flexural hinge is calculated using the axial force-moment interaction diagram and the column axial load due to gravity loads.

Flexural hinges are assigned at column ends. Additional hinges are assigned at intermediate levels when beams are connected at intermediate levels of a column or partial infills are present. Whenever necessary, induced shear forces on columns are checked with the shear capacities of the relevant components.

4.5.2 Masonry Infill Panels

At low levels of an in-plane lateral force, the frame and infill panel act in a fully composite fashion; as a structural wall with boundary elements. When lateral deformations increase, the behavior becomes more complex as a result of the frame attempting to deform in a flexural mode while the panel attempting to deform in a shear mode. The result is a separation between frame and panel at the corners on the tension diagonal, and the development of a diagonal compression strut on the compression

diagonal. In this analysis, infill panels are modeled with equivalent strut as per FEMA-273 [0] guidelines.

Failure of a masonry infill is complex and involves combination of bed joint sliding, corner crushing, and diagonal cracking. The exact mode of failure depends upon material properties such as compressive strength (f_m'), shear strength (τ_i), and friction coefficient (μ).

4.5.2.1 Equivalent Strut Model

The stiffness contribution of the infill is represented with an equivalent compression only strut connecting windward upper and leeward lower corners of the infilled frame. In this analytical model the thickness and the modulus of elasticity of the strut are assumed to be the same as those of the infill. The equivalent strut width (a) is determined using the recommended equation given in FEMA-273 [0].

$$a = 0.175[\lambda_1 h_{col}]^{-0.4} r_{inf} \quad (11)$$

$$\lambda_1 = \left[\frac{E_{me} t_{inf} \text{Sin}(2\theta)}{4 E_{fc} I_{col} h_{inf}} \right]^{1/4} \quad (12)$$

Where h_{col} = height of column, E_{me} = modulus of elasticity of masonry infill wall, t_{inf} = thickness of infill wall, θ = aspect ratio of infill wall, E_{fc} = modulus of elasticity of reinforced concrete frame, I_{col} = moment of inertia of reinforced concrete column, and h_{inf} = height of infill wall.

Since it is difficult to give a good representation of infill panel behavior under seismic forces, the experimental studies of the unreinforced masonry shear strength may give greater shear strength than true shear strength of masonry under cyclic loading. By considering above factors and the available test results the equation proposed by Paulay and Priestley [0] for shear strength of uncracked masonry is,

$$\tau_i = \tau_o + \mu f_m \quad (13)$$

where: f_m is the compressive stress of infill due to gravity load. For typical range of axial compression stresses,

$$\tau_o = 0.04 f_m' \quad (14)$$

Infill panels do not have a tight connection with the overlying beam. The vertical extension of tension column tends to separate the frame and the panel along the top edge. Hence, there is no contribution from the term μf_m in Eq. 13 [0]. The horizontal component of the force resisted by the equivalent strut should be compared with the expected shear

strength of the infill panel. Hence, the expected infill shear strength is,

$$V_{ine} = A_{ni} \tau_i \quad (15)$$

4.5.2.2 Load Deformation Relationship

FEMA-273 [0] recommends a nonlinear simplified load-deformation relations for masonry infill panels in the form of story drift ratio. In this analysis effects of partial infill panels are also considered. Since there is no well-established guidelines for modeling of perforated infills, the strut formation is assumed as shown in Figure 5. Shear strength of a panel with opening is considered in proportion to the ratio of the *infilled area/total area* of the panel. The ratio h_{inf}/l_{inf} is assumed to be one. Other modeling parameters are considered in accordance with FEMA-273 [0].

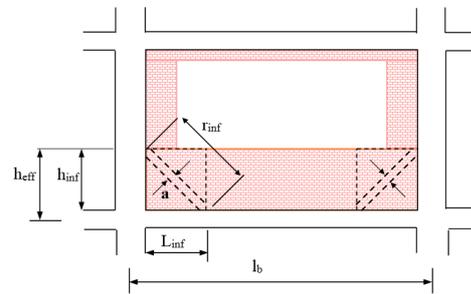


Figure 5: Infill panel with opening

4.5.3 Foundation Model

Foundation is modeled applying the Winkler model concept. This building has two types of foundations, single pile and double-pile, both of the foundation systems do not have dowels connecting the pile and the pile cap. There is no possibility to transfer any moment from column base to the pile. Thus, the foundation was modeled as pin supports. For this model, the column bases are evaluated for the resulting axial and shear forces as well as the ability to accommodate the necessary end rotations of the columns.

5.0 Summary and Conclusions

A 5-story reinforced concrete building is chosen as a representative of typical low-rise reinforced concrete commercial buildings. This building has typical irregular form of structural system found in many parts of the world. Moreover, a poor foundation system and several partial infill masonry panels are some other undesirable features found in this building system. Therefore, to make a realistic seismic evaluation of the building, a complex nonlinear FEM model was developed.

The selected building was analysed to evaluate its seismic performances during the ground motions

having 2%, 10%, and 50% of probability of being exceeded in a 50-year period. It was possible to see the effects of two-dimensional and three-dimensional modeling, lateral loading pattern, foundation, and nonstructural components on failure mechanisms. Each and every influential parameter was studied and updated the model step by step to accommodate all the parameters for the seismic evaluation. Finally, three strengthening methods were proposed to overcome the deficiencies in the existing structure.

The following conclusions are derived from this study:

- Existing structure has low level of seismic capacity and may cause the most undesirable brittle failure.
- A lot of earthquake vulnerable features, such as weak/soft story, strong beam/weak column phenomena, torsion, and $P-\Delta$ effects can be found.
- There is not much effect of partial infill panels and foundation on the seismic capacity of this building.
- Though very few irregularities can be identified, they change the structural behavior significantly depending on the analysis methods, 2D or 3D. Some irregularities, which are dominant in 2D analysis, are not effective in 3D analysis.
- Since actual structural behavior is three-dimensional, the most realistic 3D nonlinear FEM model is needed for seismic performance evaluation.
- For the improvement of existing structures, it is possible to test and evaluate many strengthening schemes. Their effectiveness can be clearly seen and quantified. The most cost-effective scheme can be easily identified. For this particular building, it is found that the best way is to improve the deformability of the structure while removing other earthquake vulnerable features of the existing structure. But strengthening of the superstructure is limited by the poor foundation system.

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