Assessment of Robustness of Masonry Infilled RC Frame Buildings with Consideration of Irregularities

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SUMMARY:
Presence of irregularities in reinforced concrete (RC) buildings increases seismic vulnerability and may lead to disproportionate collapse of such buildings. In order to minimize the likelihood of disproportionate collapse during earthquake, RC buildings should be robust. Robustness is a desirable property of structural systems which mitigates their susceptibility to disproportionate collapse. In this paper, the effects of irregularity parameters, namely, soft storey, weak storey and poor construction quality on the robustness of a six-storey three-bay RC frame were quantified. Nonlinear finite element analysis of the frame is performed and parametric study is undertaken to generate a predictive equation through response surface method. It was observed that the irregularities have significant influence on the robustness. A general observation on the implication of these irregularities and robustness is developed.

Keywords: Masonry infill RC frame building, Robustness, Irregularities

1. INTRODUCTION

Disproportionate collapse and damage of reinforced concrete (RC) and unreinforced masonry (URM) buildings is reported from past earthquakes (2001 Bhuj, 2010 Haiti earthquake). For functional reasons, the URM are often placed in upper floors, and subsequently, rendering the RC buildings irregular. The irregularities are mainly vertical irregularities (e.g., soft storey (SS), weak storey (WS), short-column effect (SCE)). Vulnerability of RC buildings in presence of these irregularity parameters has been studied by Tesfamariam and Saatcioglu (2008, 2010). In order to minimize the likelihood of progressive failure during earthquake shaking, RC buildings should be robust. Robustness is a desirable property of structural systems which helps to mitigate their susceptibility to disproportionate collapse. It is defined as the insensitivity of a structure to local failure. It is strongly related to the internal structural characteristics such as redundancy, ductility, load distribution, and joint behaviour characteristics in structure and also depends on consequences of failure. The definition of robustness varies significantly. Starossek and Haberland (2010) presents a list of such definitions available in literature and design standards.

Past seismic performance of buildings underline the need for quantifying robustness of buildings with the consideration of the irregularities. In general the common desirable parameters of the design for earthquake load and for robust structure are the ductility and good configuration of the structure to have alternate loading path in damage state. In this paper, the effects of SS and WS and interaction on the robustness of RC frame were studied for a six-storey three-bay frame. Nonlinear finite element analysis of the frame is performed and the response surface method is used to develop a predictive equation for robustness as a function of the irregularity parameters. A full factorial design of experiments (DOE) method, aimed at finding a functional description of how factors affect the response, is considered. More specifically, the interest is in exploring the main and possible interaction effects of the performance modifiers (also referred to as design factors) on a main performance indicator (response) of an RC building under earthquake loads. It is believed that similar applications of the DOE methods, especially once combined with computer experiments, can be beneficial for quantifying robustness of structures. The concept of DOE has been previously used in other earthquake engineering applications (e.g., Verderame et al. 2010; Iervolino et al. 2007; Zhang and Foschi, 2004).

2. ROBUSTNESS IN STRUCTURES

Many modern building codes such as JCSS (JCSS, 2002), national building code of Canada (NRCC, 2010), Eurocode (CEN, 1994), ASCE standard (ASCE, 2002) specify the need for the structural robustness in the sense that the failure consequences should not be disproportional to the effect of causing it. An overview of these code provisions is presented by Ellingwood (2002). However, most of these codes provide only qualitative description of the robustness and do not specify the quantitative measurement of robustness in structures and the minimum acceptable limit of robustness. Recently, several researchers (Baker et al., 2008; Starossek and Haberland, 2011) presented frameworks for the measurement of robustness. These methods can be broadly classified as deterministic, probabilistic and risk-based quantification approaches.

Frangopol and co-authors (Frangopol and Curley, 1987; Fu and Frangopol, 1990; Biondini et al., 2008) proposed probabilistic measures of structural redundancy, based on the relation between damage probability and system failure probability. Redundancy is also closely related to the level of robustness since redundant systems are generally believed to be more robust. Lind (1995, 1996) proposed a measure of system damage tolerance based on the increase in failure probability due to the occurrence of damage. Ben-Haim (1999) quantified robustness using information-gap theory.

Baker et al. (2008) proposed risk-based measurement of robustness by comparing risk associated with direct consequence of potential damages to the system, and indirect consequences corresponding to the increased risk of a damaged system.

ISO-19902:2007 (ISO, 2007) specifies the deterministic approach to obtain robustness for offshore platform. In this approach, the ratio of the base shear capacity and design load corresponding to the ultimate collapse is used as the robustness measure. In this approach, the base shear capacity of structure, with and without a particular structural element, is compared to obtain the robustness index. Starossek and Haberland (2008, 2011) proposed stiffness, damage, and energy based robustness indices. In stiffness-based approach, the stiffness matrices of the undamaged structure and that after removal of a structural element are compared. In damage based approach, the maximum damage progression caused by the "assumed initial damage" and "acceptable damage progression" are compared to obtain the robustness index (Starossek and Haberland, 2008). Similarly, in energy-based approach, the energy released by initial failure of a structural element and the energy required for the failure of the subsequently affected structural element are used to quantify robustness of structure. Izzuddin et al. (2008) proposed a multi-level framework for progressive collapse assessment of building structures due to sudden column losses. The framework employs three stages, namely, determination of the nonlinear static response, simplified dynamic assessment and ductility assessment.

In the present study, the stiffness base deterministic approach is used to measure the robustness of the six-storey three-bay RC frame. The existing stiffness-based method is based on the consideration of removing structural members (e.g. Starossek and Haberland 2008, 2011). For consideration of irregularities and seismic performance, the existing stiffness-based method is modified as:

$$r_f = \frac{k_{75}}{k_i} \tag{2.1}$$

where k_i is the initial stiffness of the infilled frame and k_{75} is the stiffness of the damage frame corresponding the 75% of the base shear capacity of the RC frame.

3. BUILDING DESIGN CONSIDERATION, FINITE ELEMENT MODELLING AND ANALYSIS

A six-storey three-bay RC frame, as shown in Fig. 3.1, was considered for the present study. The building is designed as per the National Building Bode of Canada (NRCC, 2010) using the capacity design concept with strong column-weak beam principle. For modelling the frame, a nonlinear finite element analysis is performed in open-source program OpenSees (Mazzoni et al., 2006). The details of the modelling of frames and infill wall are described in the following.

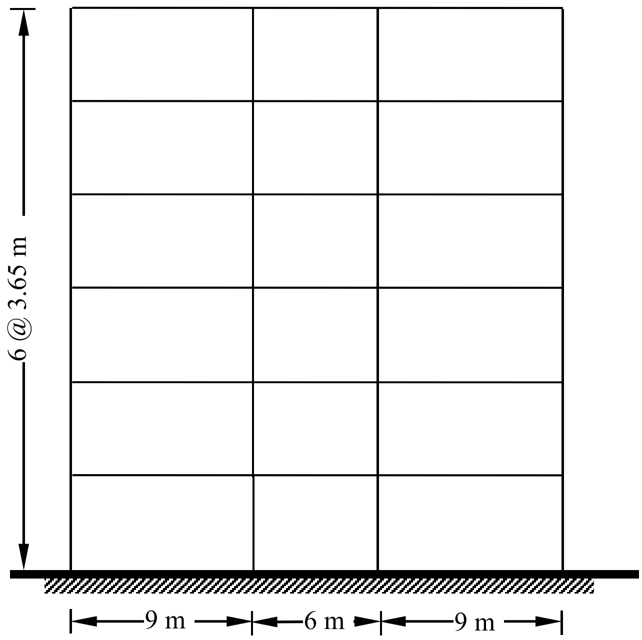

Figure 3.1. Details of the six-storey three-bay building frame considered in this study

3.1. Modelling of RC Frame

Each beam and column of the RC frame is modelled using a single beam with hinges element available in OpenSees. The cross sections of these elements are discretized into fibers of confined concrete (core), unconfined concrete (cover) and reinforcing steel. The uniaxial Kent-Scott-Park constitutive model with no tensile strength is used to model the concrete material (Kent and Park, 1971). The loading-unloading rules suggested by Karsan and Jirsa (1969) are adopted for the hysteresis behaviour of the concrete stress-strain relation in the compression region. Confinement model proposed by Braga et al. (2006) is used to quantify the effect of transverse steel. Giuffré-Menegotto-Pinto steel constitution model is used to represent the reinforcing steel behaviour (Giuffré and Pinto, 1970). P-Δ effect is not included in this analysis. The fundamental period of the RC fully infilled frame was obtained as 0.4s.

3.2. Modelling of Infilled Masonry

The method of element removal technique proposed by Kadysiewski and Mosalam (2009) is used to model the masonry infill. This model considers the interaction of in-plane (IP) and out-of-plane (OP) effects. Salient features of this model are discussed further below. Detailed discussion can be found elsewhere (Kadysiewski and Mosalam, 2009).

3.2.1. Model consideration

Each infill panel is modelled as a single diagonal strut as specified in FEMA-356 (FEMA, 2000) and comprised of two equal-size beam with hinges elements connected at the midpoint node with OP mass as shown in Fig. 3.2. The OP mass at the midpoint span node is calculated as 81% of the total mass of the infill wall panel (Kadysiewski and Mosalam, 2009). The inelastic fiber section is assigned to the ends of the elements connected to the midpoint node. Elastic sections with very small moment of inertia are assigned to the ends attached to the surrounding frame to simulate moment release. The hinge length near the mid-span node is selected as 1/10 of the total length of the diagonal strut to produce a relatively sharp yield point for the element, while at the same time providing a numerically stable solution. The thickness (normal to the wall) of the strut is equal to the actual infill thickness and the width of the strut is given by Eqn. 7-14 in FEMA 356 (FEMA, 2000). The OP strength of the URM infill wall is determined using the procedure in Section 7.5.3.2 of FEMA 356 (FEMA, 2000).

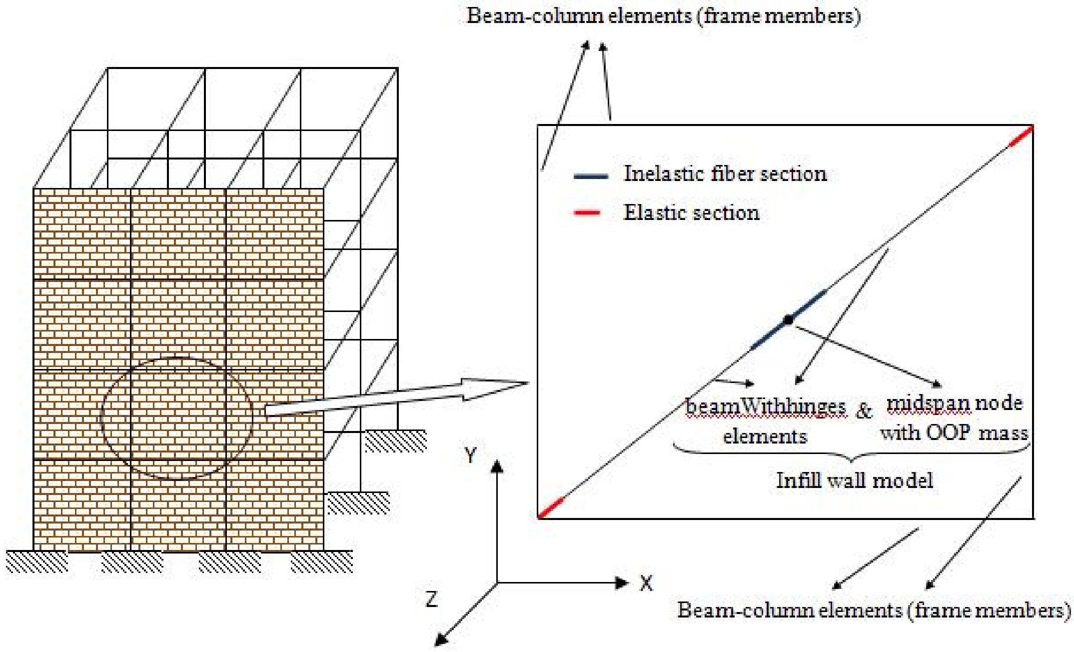

Figure 3.2. Infill wall model (adopted from OpenSees (Mazzoni *et al.*, 2006))

The following IP and OP interaction equation proposed by Kadysiewski and Mosalam (2009) is considered to obtain the properties of fiber section for the masonry infill.

$$\left(\frac{P_{IP}}{P_{IPO}}\right)^{3/2} + \left(\frac{M_{OP}}{M_{OPO}}\right)^{3/2} = 1.0 \quad (3.1)$$

where P_{IP} is the IP axial strength in presence of OP force, and P_{IPO} is the IP axial strength without OP force. M_{OP} is the OP bending strength in the presence of IP force, and M_{OPO} is the OP bending strength without IP force.

The element removal technique works based on the interaction between the IP and OP displacements. IP displacement is the relative horizontal displacement between the top and bottom nodes of the diagonal element. OP displacement is that of the middle node of the strut with respect to the chord connecting the top and bottom nodes. The following equation similar to strength interaction in Eq. (3.1) is considered for the IP and OP displacement of masonry infill:

$$\left(\frac{\Delta_H}{\Delta_{HyO}}\right)^{3/2} + \left(\frac{\Delta_N}{\Delta_{NyO}}\right)^{3/2} = 1.0 \tag{3.2}$$

where Δ_H is the IP horizontal displacement, Δ_N is the OP horizontal displacement, and Δ_{HyO} and Δ_{NyO} are their respective yield values at zero load in the opposite direction.

3.2.2. Material properties

Unconfined compressive strength of concrete (f_c') is taken as 30 MPa. Modulus of elasticity of concrete (E_c) is taken as 27,400 MPa. The Poisson's ratio and mass density of concrete are taken as 0.2, and 2,500 kg/m3, respectively. HYSD steel bars of the yield strength (f_y) of 400 MPa is used for the reinforcement. Compressive strength of masonry (f_m') is taken as 17 MPa and the modulus of elasticity of masonry (E_m) has been obtained from the following equation (FEMA, 2000):

$$E_m = 550f_m' \tag{3.3}$$

4. CONSIDERATION OF IRREGULARITIES AND PARAMETRIC STUDY

4.1. Soft Storey (SS)

Soft storey is defined by stiffness of lateral force resisting system in any storey being less than 70% of the stiffness of an adjacent storey (above or below) or less than 80% of the average stiffness of the three stories (above or below) (FEMA, 1998, 2004). Further, *FEMA 450-1* (2004) specifies that extreme soft storey is said to exist when lateral force resisting system in any storey is less than 60%. The storey stiffness (k_w) has been obtained from the following equations considering no interaction between column and wall stiffness (Gulkan and Sozen, 1999):

$$k_{story} = \frac{c_c E_c p_c A_f}{h_s \lambda^2} + \frac{E_m p_w A_f}{h_s \left\{\frac{(h_s/l_w)^2}{c_w} + \frac{2.5}{c_{wa}}\right\}} \tag{4.1}$$

where h_s is the storey height, c_c is a constant which depends on the end fixity of column, c_w is the constant depending on wall end conditions and masonry cross section properties related to bending, c_{wa} is a constant depending on masonry cross section properties related to shear distortions, p_c and p_w are the column and wall ratio, respectively, A_f is the floor area, l_w is the length of the wall, λ is the slenderness ratio of column

4.2. Weak Storey (WS)

Weak storey (WS) is defined by strength of lateral force resisting system of any storey is less than 80% of the adjacent storey strength (above or below). A structure with storey strength less than 65% of the storey above is prohibited (Al-Ali and Krawinkler, 1998). The potential for weak storey can be obtained from "strength irregularity factor" (Ministry of Public Works and Settlement, 1998):

$$\eta_{ci} = \frac{(\sum A_e)_i}{(\sum A_e)_{i+1}} < 0.80 \tag{4.2}$$

where A_e is effective shear area of any storey, and the indices i and i+1 denotes the two adjacent floors.

4.3. Parametric Study

Parametric study has been conducted on RC infilled frame for three different thickness of infill (t_{infill})

(250 mm, 125 m and 0 mm) for three different tie spacing (75 mm, 150 mm and 300 mm) in beams and columns and for absence of infill at 1st and subsequent stories as shown in Table 4.1.

Pushover analysis (also called nonlinear static analysis (NSA)) is performed with force profile similar to the fundamental mode shape of the RC frame to obtain to obtain the post-yield behaviour of the structure. Displacement control pushover analysis is carried out with slowly increasing displacement (0.01 mm) in each step until a target displacement of 20% of the height of the frame is reached. The base shear is plotted as a function of the roof displacement to obtain the pushover curve (capacity curve). Pushover curve of bare frame with different tie spacing in beams and columns indicates that the stiffness of RC frame does not decrease due to increase in tie spacing (Fig. 4.1).

Table 4.1: Parameters considered for the simulation

Parameters	Infill thickness (T_{infill})	Tie spacing (s)	Number of infilled walls (N_{infill})
Very Low (VL)	-	-	0
Low (L)	250	75	1, 2
Medium (M)	125	150	1,2,3
High (H)	0	300	1,2,3,4
Very High (VH)	-	-	1,2,3,4,5
Extreme (E)	-	-	1,2,3,4,5,6

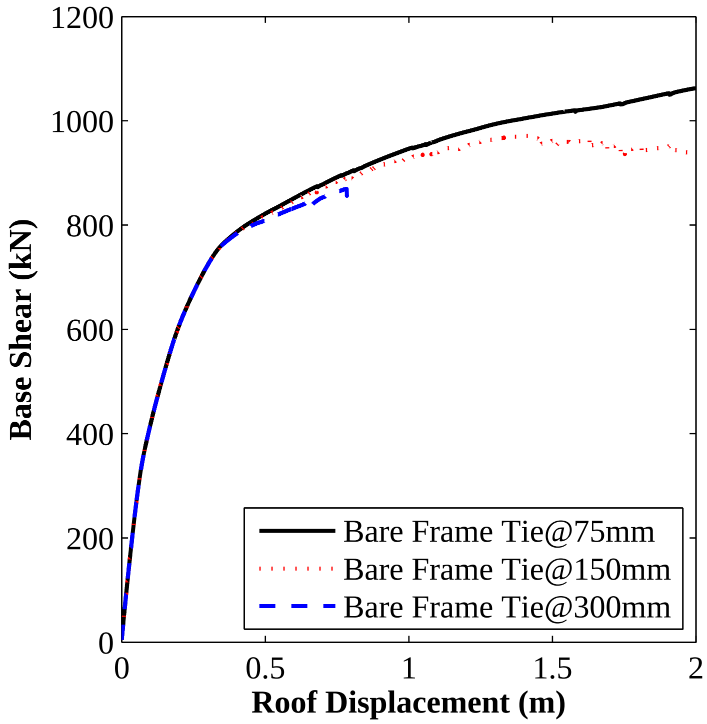

Figure 4.1. Effect of masonry panel on the stiffness of RC infilled frame

Figure 4.2 shows the pushover curve to highlight the effect of masonry panel on the stiffness of RC infilled frame. It shows that the stiffness of RC frame is significantly high as compared to the bare frame. As expected the stiffness in absence of infill at 1st two-storey is less than that of the fully infilled frame. Further removal of the infills in subsequent floors decrease the stiffness and the pushover curves moves towards that of the bare frame. For infilled frame, the masonry panels carry most of the loads until they fail. Therefore, no damage is observed in frame members till the failure of the masonry panel even in absence of infill in 1st two stories. This is because the frame is designed according to modern seismic code with strong-column weak beam concept. However, for bare frame, the decrease in stiffness was observed at base shear of about 300 kN due to the damage in columns and beams. As expected, both the strength and stiffness increase due to increase of thickness of infill. Because of the large thickness, the infill damage occurred at the beams and columns prior to the significant damage of infill.

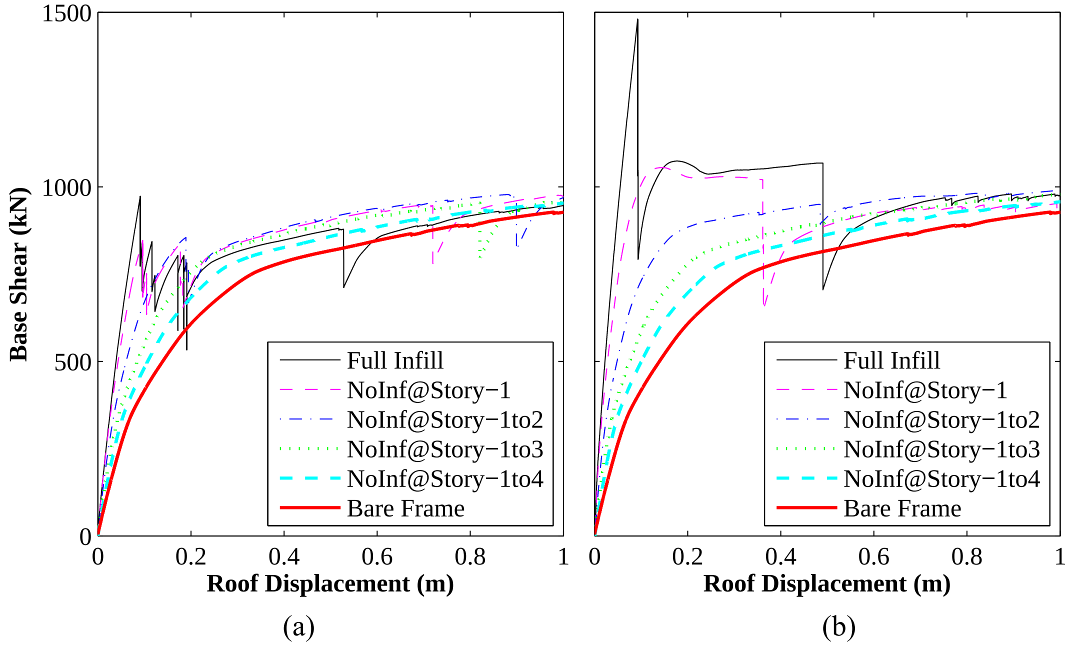

Figure 4.2. Effect of masonry panel on the stiffness of RC infilled frame with wall thickness (a) 125 mm and (b) 250 mm

The results for each simulation are summarized in Table 4.2. The parameters K_{storey} and WS have been obtained using Eqns. 3.4 and 3.5, respectively. The parameter K_{frame} is obtained by combining stiffness of each storey connected in series. The parameter K_i is the initial stiffness obtained from the pushover analysis in OpenSees. The robustness index, r_f, is obtained from the pushover analysis using Eqn. 2.1. These results in Table 4.2 will be used for the response surface method.

Table 4.2: Parameters considered for the simulation

Infill thickness T_{infill}	WS	Number of infilled walls N_{infill}	K_{frame} (kN/mm)	k_{storey} (kN/mm)	K_i (kN/mm)	r_f
0	100	0	426	2557.8	13.00	0.88
0	25	1	140	199	11.89	0.77
0	25	2	85	199	8.77	0.68
0	25	3	61	199	6.39	0.51
0	25	4	48	199	5.10	0.51
0	25	5	39	199	4.32	0.51
-1	25	6	33	199	3.92	0.53
1	100	0	819	4916	23.09	0.76
1	14	1	166	199	18.82	0.71
1	14	2	92	199	11.16	0.61
1	14	3	64	199	7.25	0.50
1	14	4	49	199	5.46	0.48

5. PREDICTIVE EQUATIONS USING RESPONSE SURFACE METHOD

Response surface methods (RSM) is a collection of mathematical and statistical techniques for solving problems in which the goal is to optimize the response y of a system or process using n independent variables, subject to observational errors (Montgomery, 2011). Response surfaces are smooth analytical functions and are most often approximated by linear function (first order model) or polynomial of higher degree (such as the second-order model). The second-order polynomial response surface has the form:

$$y = \beta_O + \sum_{i=1}^{n} \beta_i x_i + \sum_{i=1}^{n} \beta_{ii} x_i^2 + \sum_{i=1}^{n} \sum_{j=1}^{i} \beta_{ij} x_i x_j \tag{5.1}$$

where y is regression equation, and β_0, β_i and β_{ij} are the regression coefficients. Estimates of the coefficients β_0, β_i and β_{ij} can be obtained by fitting the regression equation to the response surface values observed at a set of data points. For a second order response surface, $(n+1)(n+2)/2$ unknown regression parameters are present and in order to estimate these parameters, an equal number of data points are needed. Different authors have reported generation of response surface method in reliability engineering (Faravelli, 1989; Rajashekhar and Ellingwood, 1993; Pinto, 2001; Möller et al., 2009; Buratti et al., 2010).

Initial sensitivity analysis showed that, the first order response surface for K_i and r_f is deemed to be adequate. Thus K_i and r_f values are quantified in terms of thickness of infill (T_{infill}), WS, number of infilled walls (N_{infill}), stiffness of frames (K_{frame}), and stiffness of storey (K_{storey}) as:

$$K_i \; or \; r_f = \beta_0 + \beta_1 T_{infill} + \beta_2 WS + \beta_3 N_{infill} + \beta_4 K_{frame} + \beta_5 K_{story} \tag{5.2}$$

The regression coefficients, $\beta_{i\,(i=1,5)}$ corresponding t statistics (t Stat) are computed and summarized in Table 5.1. It can be observed that importance of each factor that can be inferred from t Stat. From the t Stat, the K_i parameters are dominated by K_{frame}, K_{story} and T_{infill}. Whereas, the r_f is dominated by K_{frame}, K_{story} and WS.

Table 5.1. Response surface regression coefficients for K_i and r_f

Coefficients	Parameter K_i (R^2 = 0.98)		Parameter r_f (R^2 = 0.95)	
	Value	t Stat	Value	t Stat
β_o	-0.37	-0.08	0.4618	3.01
β_1	1.49	1.39	-0.0281	-0.75
β_2	0.03	0.52	0.0049	2.15
β_3	0.57	0.85	-0.0165	-0.70
β_4	0.11	5.01	0.0019	2.39
β_5	-0.01	-3.60	-0.0003	-2.40

6. CONCLUSIONS

Robustness is a desirable property of structural systems which mitigates their susceptibility to disproportionate collapse. In the paper, the effect of irregularity parameters, namely, soft storey, weak storey and the construction quality on the robustness of RC infilled frame has been studied. The robustness of the RC frame has been quantified based on the stiffness of the RC frame. Nonlinear pushover analysis is conducted and parametric study was done considering thickness of infill, tie spacing in beams and columns, and number of infill wall as parameters. It was observed that the stiffness of infilled frame is insensitive to the tie spacing. Using the results of the parametric study, a predictive equation is developed for robustness as a function of the irregularity parameters. It was observed that the initial stiffness of RC infilled frame is dominated by the stiffness of the frame (excluding infill), story stiffness and thickness of infill. Whereas, robustness of the RC frame is dominated by the stiffness of the frame, story stiffness and weak storey.The predictive equation should be modified in future considering more number of parameters.

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