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QUALITATIVE REVIEW OF SEISMIC RESPONSE OF VERTICALLY IRREGULAR BUILDING FRAMES

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ABSTRACT

This study summarizes state-of-the-art knowledge in the seismic response of vertically irregular building frames. Criteria defining vertical irregularity as per the current building codes have been discussed. A review of studies on the seismic behavior of vertically irregular structures along with their findings has been presented. It is observed that building codes provide criteria to classify the vertically irregular structures and suggest dynamic analysis to arrive at design lateral forces. Most of the studies agree on the increase in drift demand in the tower portion of set-back structures and on the increase in seismic demand for buildings with discontinuous distributions in mass, stiffness, and strength. The largest seismic demand is found for the combined-stiffness-and-strength irregularity.

KEYWORDS: Mass Irregularity, Set-back Structure, Stiffness Irregularity, Strength Irregularity, Vertical Irregularity

INTRODUCTION

Irregular buildings constitute a large portion of the modern urban infrastructure. The group of people involved in constructing the building facilities, including owner, architect, structural engineer, contractor and local authorities, contribute to the overall planning, selection of structural system, and to its configuration. This may lead to building structures with irregular distributions in their mass, stiffness and strength along the height of building. When such buildings are located in a high seismic zone, the structural engineer's role becomes more challenging. Therefore, the structural engineer needs to have a thorough understanding of the seismic response of irregular structures. In recent past, several studies have been carried out to evaluate the response of irregular buildings. This paper is an attempt to summarize the work that has been already done pertaining to the seismic response of vertically irregular building frames.

CRITERIA FOR VERTICAL IRREGULARITIES IN BUILDING CODES

In the earlier versions of IS 1893 (BIS, 1962, 1966, 1970, 1975, 1984), there was no mention of vertical irregularity in building frames. However, in the recent version of IS 1893 (Part 1)-2002 (BIS, 2002), irregular configuration of buildings has been defined explicitly. Five types of vertical irregularity have been listed as shown in Figure 1. They are: stiffness irregularity (soft story), mass irregularity, vertical geometric irregularity (set-back), in-plane discontinuity in lateral-force-resisting vertical elements, and discontinuity in capacity (weak story).

NEHRP code (BSSC, 2003) has classifications of vertical irregularities similar to those described in IS 1893 (Part 1)-2002 (BIS, 2002). As per this code, a structure is defined to be irregular if the ratio of one of the quantities (such as mass, stiffness or strength) between adjacent stories exceeds a minimum prescribed value. These values (such as 70-80% for soft story, 80% for weak story, 150% for set-back structures) and the criteria that define the irregularities have been assigned by judgment. Further, various building codes suggest dynamic analysis (which can be elastic time history analysis or elastic response spectrum analysis) to come up with design lateral force distribution for irregular structures rather than using equivalent lateral force (ELF) procedures.

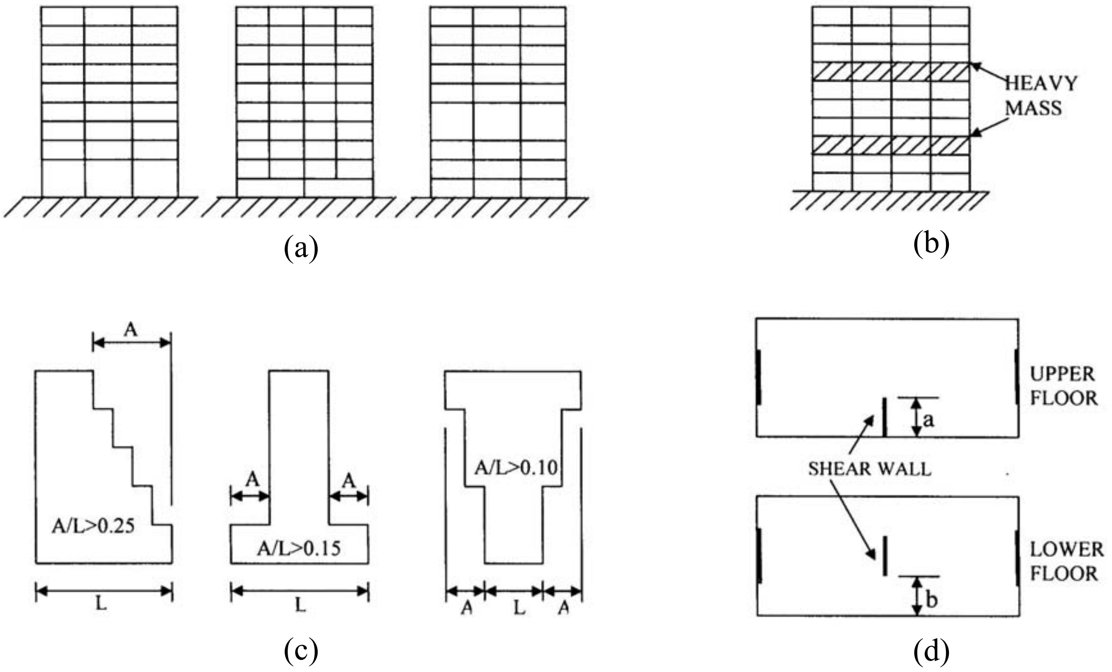

Fig. 1 (a) Stiffness/strength irregularity; (b) Mass irregularity; (c) Vertical geometric irregularity or set-back; (d) In-plane discontinuity in lateral-force-resisting vertical elements when $b > a$: plan view (after BIS, 2002)

REVIEW OF PREVIOUS STUDIES ON VERTICAL IRREGULARITY

The seismic response of vertically irregular building frames, which has been the subject of numerous research papers, started getting attention in the late 1970s. A large number of papers have focused on plan irregularity resulting in torsion in structural systems. Vertical irregularities are characterized by vertical discontinuities in the distribution of mass, stiffness and strength. Very few research studies have been carried out to evaluate the effects of discontinuities in each one of these quantities independently, and majority of the studies have focused on the elastic response. There have also been detailed studies on real irregular buildings that failed during earthquakes (Mahin et al., 1976; Kreger and Sozen, 1989), but such studies are small in number. Many researchers studied the response of set-back structures (Humar and Wright, 1977; Aranda, 1984; Moehle and Alarcon, 1986; Shahrooz and Moehle, 1990; Wong and Tso, 1994). In set-back structures there is a sudden change in the vertical distribution of mass, stiffness, and in some cases, strength. A set-back structure is thought of being made up of two parts: a base (the lower part having many bays), and a tower (the upper part with fewer bays). Following is a brief review of the work that has been done on the seismic response of set-back structures.

Humar and Wright (1977) studied seismic response of steel frames with set-backs by using one ground motion. They found story drifts to be larger in the tower parts of set-back structures than those for the regular structures. On the other hand, smaller story drifts were found in the base parts of set-back structure as compared to the regular structures. They concluded that the difference in elastic and inelastic story drifts between set-back and regular structures depends on the level of story considered. Most notable observations were altered displacements and high ductility demands in the vicinity of the irregularities.

Aranda (1984) made a comparison of ductility demands between set-back and regular structures by using ground motions recorded on soft soil. He observed higher ductility demands for set-back structures than for the regular ones and found this increase to be more pronounced in the tower portions.

Shahrooz and Moehle (1990) observed based on their analytical study that damage is concentrated in the tower portion of a set-back structure due to high rotational ductilities. They also performed experimental studies and concluded that fundamental mode dominates the response in the direction parallel to the set-back.

During the experimental study by Wood (1992) on two models of set-back frames, she noticed that the response of set-back structures did not differ much from that of the regular structures.

Wong and Tso (1994) studied the response of set-back structures by using elastic response spectrum analysis. They observed that the modal masses of higher modes are larger for the set-back structures resulting in different seismic load distributions as compared to those from the static code procedure.

From the above studies on set-back structures, it can be seen that there are varied conclusions regarding the response of set-back structures.

Moehle and Alarcon (1986) carried out an experimental response study on two small scale models of reinforced concrete frame-wall structures subjected to strong base motions by using shake table. One of the test structures, designated as FFW, had two nine-story, three-bay frames and a nine-story, prismatic wall. The other structure, designated as FSW, was identical to FFW except that the wall extended only to the first floor level. Thus the test structures FFW and FSW represent the buildings having "regular" and "irregular" distributions of stiffness and strength in vertical plane respectively. They compared the measured response with that computed by the inelastic dynamic response time-history analysis, inelastic static analysis, elastic modal spectral analysis, and elastic static analysis. Several inelastic response time-history analyses were conducted for each test structure. For each analysis, different modeling assumptions were tried in an effort to establish a "best-fit" model. They compared maximum top-floor displacements obtained by the experiments and by different inelastic dynamic and elastic analysis methods. One such comparison is shown in Table 1. It shows that the best estimates of maximum displacement are obtained via "Analysis B" and "Analysis C" of the inelastic dynamic analyses (see Table 1 for details on the two models). Thus they concluded that the main advantage of dynamic methods is that those are capable of estimating the maximum displacement response, whereas the static methods cannot be used for this purpose. Further, they inferred that the inelastic static and dynamic methods are superior to the elastic methods in interpreting the structural discontinuities.

Table 1: Maximum Top-Floor Displacements (mm)

Structure	Measured	Inelastic Dynamic Analyses			Elastic Analyses	
		"Analysis A"	"Analysis B"	"Analysis C"	Modal Spectral	Static
FFW	26.1	16.3 (0.62)	27.5 (1.05)	23.0 (0.88)	17.6 (0.67)	17.7 (0.68)
FSW	22.4	17.0 (0.76)	24.2 (1.08)	19.8 (0.88)	17.7 (0.79)	17.9 (0.80)

Values in parentheses are ratios between the calculated and measured maximum displacements.
"Analysis A" was based on the computed member moment-rotation behavior without including the effects of reinforcement slip.
"Analysis B" included slip of reinforcement at the base of walls and slip of beam reinforcement from the beam-column joints.
"Analysis C" refers to beam fixed-end rotations due to slip, reduced by computing the fixed-end rotational stiffness for bar stress levels equal to approximately half the yield stress.

Ruiz and Diederich (1989) studied the seismic performance of buildings with weak first story in case of single ground motion. They studied the influence of the lateral strength discontinuity on ductility demand at the first story under the action of the acceleration record with largest peak ground acceleration, as obtained on soft soil in Mexico City during the Mexico earthquake of September 19, 1985. A parametric study was carried out for 5- and 12-story buildings with weak first story, and with brittle infill wall in upper stories in some cases and ductile in others. The fundamental periods of these buildings were 0.67 and 1.4 s respectively. They noted that the behavior of weak first story buildings greatly depends on the ratio of the dominant periods of excitation and response, the resistances of upper and first stories, and on the seismic coefficient used for design. The ratio of dominant periods of response and excitation was found to be closely related to the formation of plastic hinges, yielding or failure of infill walls, and to the times of their occurrences.

Nassar and Krawinkler (1991) evaluated seismic demand parameters for bilinear and stiffness-degrading single-degree-of-freedom (SDOF) systems and three types of multi-degree-of-freedom (MDOF) structures of 3-, 5-, 10-, 20-, 30-, and 40-story heights and 0.217, 0.431, 0.725, 1.220, 1.653 and 2.051 s fundamental periods, respectively. The three MDOF models studied were: (a) BH (beam hinge) model, in which plastic hinges form in beams only (as well as in supports), (b) CH (column hinge) model, in which plastic hinges form in columns only, and (c) WS (weak story) model, in which plastic hinges form in columns of the first story only. They used 36 strong ground motions, recorded during single earthquake, namely, the Whittier Narrows earthquake of October 1, 1987, in and around Los Angeles, California, and 15 strong ground motions from different Western U.S. earthquakes, recorded on firm soil. In the study on SDOF models, the inelastic strength and cumulative damage demands were evaluated statistically for specified target ductility ratios. Strength demands were represented in terms of inelastic strength demand spectra or spectra of strength reduction factors. Expressions were developed that relate the strength reduction factor to period and target ductility ratio. In the study on MDOF models, they found that the required strengths for specified target ductility ratios depend strongly on the type of failure mechanisms that develop during severe earthquakes. They observed that weak first story leads to large amplifications in ductility and overturning moment demands. This has been confirmed later by the study of Seneviratna and Krawinkler (1997).

Esteva (1992) studied the nonlinear seismic response of soft-first-story buildings subjected to narrow-band accelerograms. The variables covered were: number of stories, fundamental period, form of the variation of story stiffness along height, ratio of post-yield to initial stiffness, in addition to the variable of primary interest, i.e., factor r expressing the ratio of the average value of the safety factor for lateral shear at the upper stories to that at the bottom story. He used shear-beam systems representative of buildings characterized by different number of stories and natural periods as given in Table 2. The study included cases of stories with hysteretic bilinear behavior, both including and neglecting P-delta effects. The excitation was in some cases an accelerogram recorded on soft soil in Mexico City during the Mexico earthquake of September 19, 1985, and in some cases an ensemble of artificial accelerograms with similar statistical characteristics. He observed that the nature and magnitude of the influence of the ratio r on the maximum ductility demands at the first story depend on the low-strain fundamental period of the system. For very short periods those ductility demands may be reduced by about 30% when r grows from 1.0 to 3.0. For intermediate periods, ductility demands are little sensitive to r, but for longer periods those may reach the increments of 50 to 100% while r varies within the mentioned interval. He also observed that the influence of r on the response of the first story is strongly enhanced if P-delta effects are taken into account.

Table 2: Number of Stories and Fundamental Periods of Building Frames Studied by Esteva (1992)

Number of Stories	Fundamental Periods (s)
7	0.4, 0.7, 1.0
14	1.1, 1.4, 1.5
20	1.8, 2.0

Valmudsson and Nau (1997) focused on evaluating building code requirements for vertically irregular frames. The earthquake response of 5-, 10-, and 20-story framed structures with uniform mass, stiffness, and strength distributions was evaluated. The structures were modeled as two-dimensional shear buildings. The response calculated from the time-history analysis was compared with that predicted by the ELF procedure as embodied in UBC (1994). Based on this comparison, they evaluated the requirements under which a structure can be considered regular and the ELF provisions are applicable. They concluded (see Figure 2(a)) that when the mass of one floor increases by 50%, the increase in ductility demand is not greater than 20%. Reducing the stiffness of the first story by 30%, while keeping the strength constant, increases the first story drift by 20-40%, depending on the design ductility (μ) as shown in Figure 2(b). Reducing the strength of the first story by 20% increases the ductility demand by 100-200%, depending on design ductility as shown in Figure 2(c). Reducing the first story strength and stiffness proportionally by 30% increases the ductility demand by 80-200%, depending on the design ductility as shown in Figure 2(d). Thus strength criterion results in large increases in response quantities and is not consistent with the mass and stiffness requirements.

Al-Ali and Krawinkler (1998) carried out evaluation of the effects of vertical irregularities by considering height-wise variations of seismic demands. They used a 10-story building model designed according to the strong-beam-weak-column (column hinge model) philosophy and an ensemble of 15 strong ground motions, recorded on rock or firm soil during Western U.S. earthquakes after 1983, for the parametric study. The effects of vertical irregularities in the distributions of mass, stiffness and strength were considered separately and in combinations, and the seismic response of irregular structures was assessed by means of the elastic and inelastic dynamic analyses. They found that the effect of mass irregularity is the smallest, the effect of strength irregularity is larger than the effect of stiffness irregularity, and the effect of combined-stiffness-and-strength irregularity is the largest. Roof displacement is not affected by the vertical irregularity.

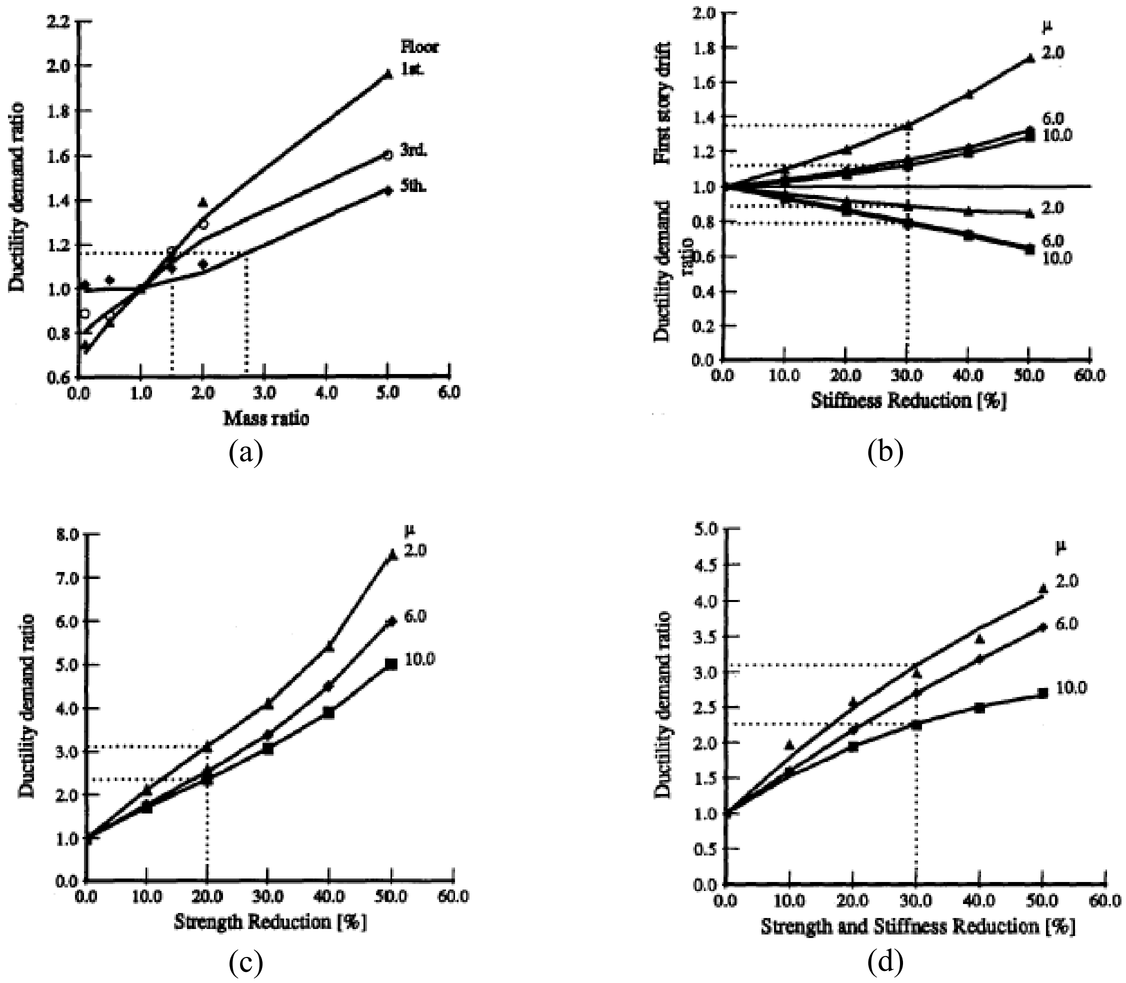

Fig. 2 (a) Maximum ductility demand for 5-story structure with mass irregularity and design ductility = 2; (b) Maximum ductility demand and first story drift for 20-story structure with stiffness irregularity; (c) Maximum ductility demand for 20-story structure with strength irregularity; (d) Maximum ductility demand for 20-story structure with strength and stiffness irregularities (after Valmudsson and Nau, 1997)

Das and Nau (2003) investigated the definition of irregular structure for different vertical irregularities: stiffness, strength, mass, and that due to the presence of non-structural masonry infill as prescribed in building codes. Linear and nonlinear dynamic time-history (TH) analyses were performed on an ensemble of 78 buildings of 5, 10, and 20 stories and with different story stiffness, strength, and mass ratios. All buildings had three bays in the direction of the ground motion. The lateral force-resisting systems considered were special moment resisting frames (SMRF) designed based on the forces obtained from the ELF procedure according to the strong-column-weak-beam (SCWB) criteria of ACI 318-99 (ACI, 1999) and UBC (1997). They observed that most structures considered in their study performed well when subjected to the design earthquake ground motion. Hence they concluded that the restrictions on the applicability of the ELF procedure given in building codes are unnecessarily conservative for certain types of vertical irregularities considered. In Figure 3(a), response of a regular structure is shown by the continuous line. Letter "A" in the legend refers to SMRF with a taller (softer and weaker) first story. The numbers following this letter represent the 'number of stories' in the structure, the 'height of the first story' (in feet), and the 'bay size' (in feet). A201525 thus represents a 20-story SMRF with a 15-feet-tall first story and 25 feet of bay size. Further, letters "t", "m", and "b" denote the location of the heavier mass: "t" for top, "m" for mid-height, and "b" for bottom. The numbers before these letters denote the 'number of stories' in the structure and the numbers following denote the 'mass ratios'. For example, 20t5 refers to a 20-story structure with a mass ratio of 5.0 on the top floor, whereas 20t25 refers to the same location of the heavier mass but with a mass ratio of 2.5 (a mass ratio of 2.5 is denoted by "25"). In Figure 3(a), ELF_R refers to the equivalent lateral force procedure that considers the 'actual' first mode shape, 'actual' fundamental period, and the corresponding 'effective mass'. This figure indicates that the response of an irregular structure designed by the ELF procedure is close to that of the regular structure. Figures 3(b) and 3(c) respectively show the variation of inelastic story drift over the heights of (i) 5-story and (ii) 10-story buildings, both having soft and weak first stories and mass irregularity. As seen from these figures, the presence of irregularity alters the inelastic response of the building, and there are marked increases in the inelastic story drift in the vicinity of the irregularity. However, in no case did the drift exceed the code-specified limit of 2%. The structure damage indices (a measure of the overall structural damage suffered by the building subjected to scaled ground motion) for all buildings were found to be less than 0.40, i.e., the threshold of repairable damage. The damage indices are insensitive to both the mass ratios and the location of the heavier mass. For all categories of the buildings studied, despite large increases on curvature ductility demands in the plastic regions in the vicinity of the irregularities, the demands did not exceed the computed curvature ductility capacities for which the members were designed. In general, it may be seen that the presence of irregularities has relatively little influence on the responses computed via ELF. This may not be true for a shorter structure, however.

Chintanapakdee and Chopra (2004) studied the effects of stiffness and strength irregularities on story drift demand and floor displacement responses. They considered 48 frames, all 12-stories high and designed according to the strong-column-weak-beam (beam hinge model) philosophy. Three types of irregularities in the height-wise distributions of frame properties were considered: stiffness irregularity (KM), strength irregularity (SM), and combined-stiffness-and-strength irregularity (KS). They studied the influence of vertical irregularities in the stiffness and strength distributions, separately and in combination, on the seismic demands of strong-column-weak-beam frames. For this, they compared the median seismic demands of irregular and regular frames computed by nonlinear time history analyses for an ensemble of 20 large-magnitude-small-distance records (LMSR). The ground motion records were obtained for earthquakes from California, with magnitudes ranging from 6.6 to 6.9, and for firm ground sites at epicentral distances of 13 to 30 km. They found that introducing a soft and/or weak story increases the story drift demands in the modified and neighboring stories and decreases the drift demands in other stories, as shown in Figure 4(a). On the other hand, a stiff and/or strong story decreases the drift demands in the modified and neighboring stories and increases the drift demands in other stories, as shown in Figure 4(b). Irregularity in upper stories has very little influence on the floor displacements, as shown in Figures 4(c) and 4(d). In contrast, irregularity in lower stories has significant influence on the height-wise distribution of floor displacements, as observed in Figures 4(e) and 4(f). These results are found to be significantly different from those reported using less realistic column hinge models by Al-Ali and Krawinkler (1998).

Fragiadakis et al. (2006) proposed a methodology based on Incremental Dynamic Analysis (IDA) to evaluate the response of structures with 'single-story vertical irregularities' in stiffness and strength using a nine-story steel frame. IDA is regarded as one of the most powerful analysis methods available, since it can provide accurate estimates of the complete range of the model's response, from elastic to yielding, then to nonlinear inelastic, and finally to global dynamic instability. IDA involves performing a series of nonlinear dynamic analyses for each record by scaling it to several levels of intensity. Each dynamic analysis is characterized by two scalars: an Intensity Measure (IM), which represents the scaling factor of the record, and an Engineering Demand Parameter (EDP), which monitors the response of the model. An appropriate choice for IM for moderate-period structures with no near-fault activity is the 5%-damped first-mode spectral acceleration $S_a(T_1,5\%)$, while a good candidate for EDP is the maximum story drift $\theta_{\max}$ of the structure. Limit-states (e.g., immediate occupancy or collapse prevention, as in FEMA-350 (FEMA, 2000)) can be defined on each IDA curve and summarized to produce the probability of exceeding a specified limit-state. The methodology proposed by Fragiadakis et al. (2006) enables a full-range performance evaluation via a highly accurate analysis method that pinpoints the effect of any source of irregularity. Fragiadakis et al. (2006) concluded that vertical irregularities produce different effects, which depend on the type of irregularity as shown in Figure 5(a), the story where it happens, and most importantly, on the intensity of the earthquake as shown in Figures 5(b) and 5(c), or equivalently on the response level or damaged state of the structure. To design an irregular structure they modified the story properties by upgrading or degrading the properties of all members of the story, i.e., the beams and the supporting columns, by a single modification factor of 2. Thus, for the case of upgraded stiffness the stiffnesses of all members of that story are multiplied by 2, while for the case of degraded stiffness, those are divided by 2. The response for upgraded and degraded cases is shown by continuous line and dashed line respectively in Figures 5(a)-5(c). Here, stiffness irregularity cases are denoted by "KI" and strength irregularity cases by "SI".

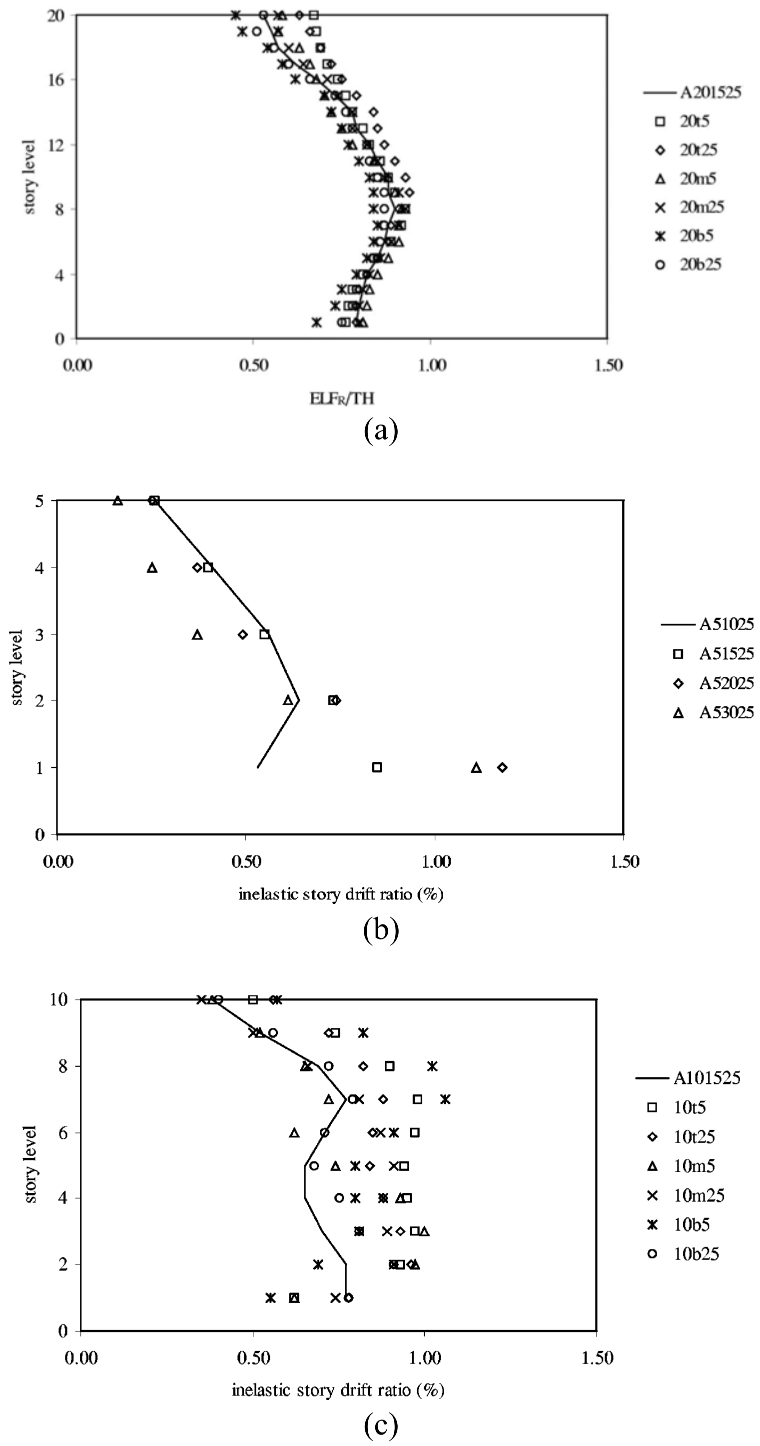

Fig. 3 (a) Ratio of elastic story drifts as predicted by ELF_R and TH for the 20-story building with mass irregularity; (b) Variation of inelastic story drift ratios for the 5-story building with mass irregularity; (c) Variation of inelastic story drift ratios for the 10-story building with mass irregularity (after Das and Nau, 2003)

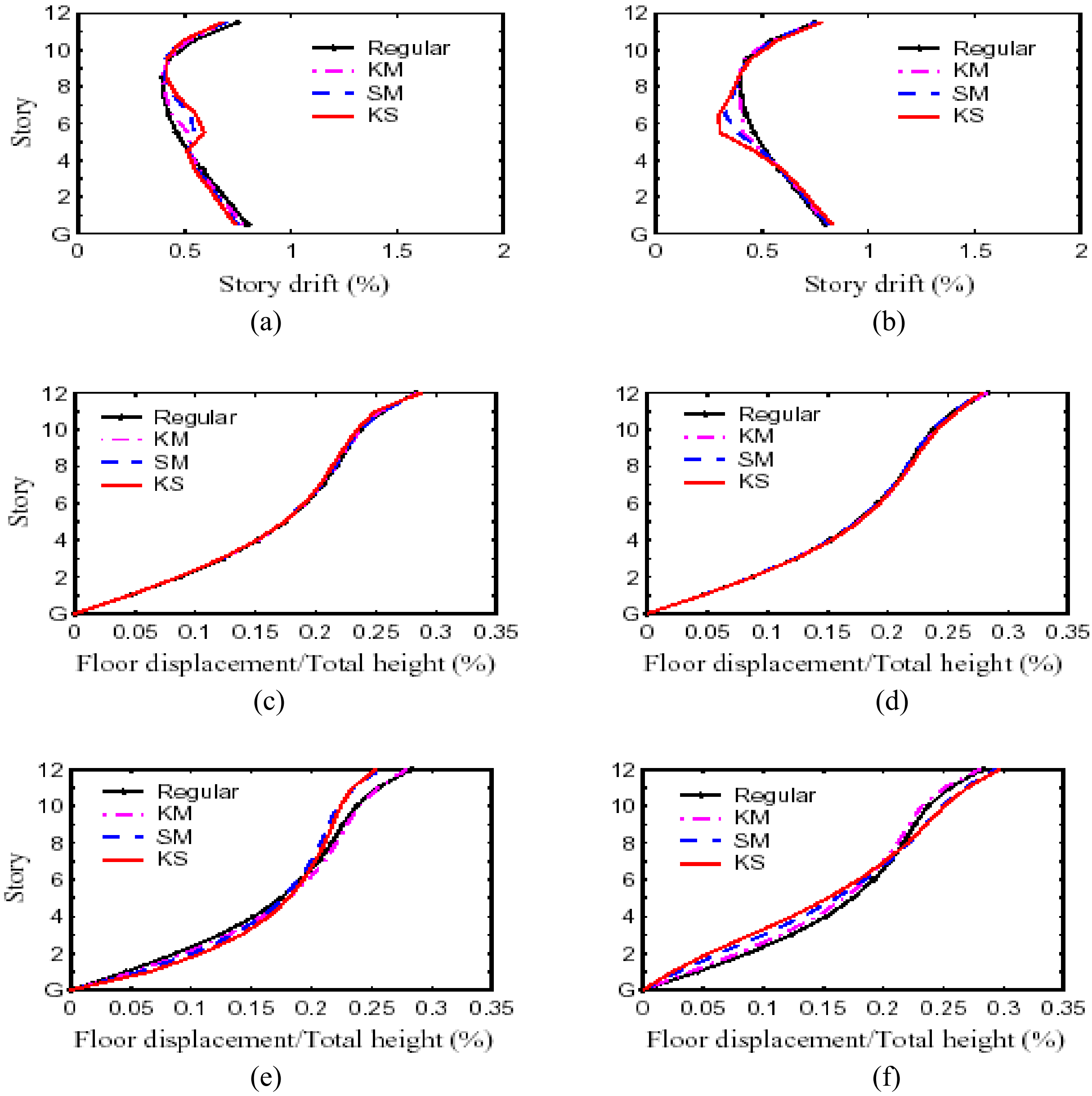

Fig. 4 (a) Ratio of story drift demand of regular and irregular frame with soft and/or weak mid-height story; (b) Ratio of story drift demand of regular and irregular frame with stiff and/or strong mid-height story; (c) Floor displacements of regular and stiffness-, strength-, and combined-stiffness-and-strength-irregular frames denoted by KM, SM, and KS, respectively, with soft and/or weak top story; (d) Floor displacements of regular and stiffness-, strength-, and combined-stiffness-and-strength-irregular frames denoted by KM, SM, and KS, respectively, with stiff and/or strong top story; (e) Floor displacements of regular and stiffness-strength-and combined-stiffness-and-strength-irregular frames denoted by KM, SM, and KS, respectively, with soft and/or weak first story; (f) Floor displacements of regular and stiffness-, strength-, and combined-stiffness-and-strength-irregular frames denoted by KM, SM, and KS, respectively, with stiff and/or strong first story (after Chintanapakdee and Chopra, 2004)

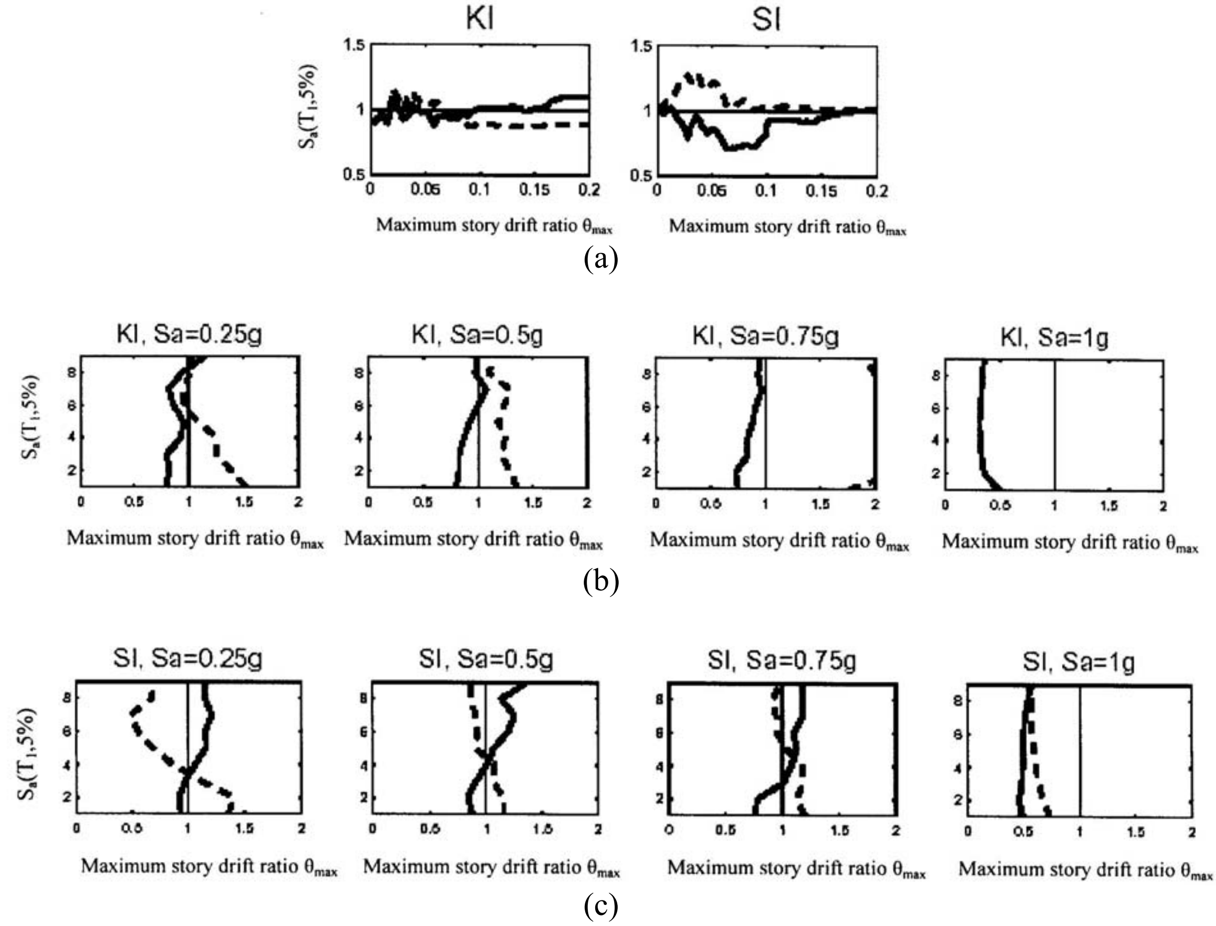

Fig. 5 (a) Spectral acceleration $S_a(T_1,5\%)$ versus θ_{max} for stiffness and strength irregularity, shown by KI and SI respectively; (b) Story drift demands for stiffness-irregular (KI) case for four $S_a(T_1,5\%)$ levels; (c) Story drift demands for strength-irregular (SI) case for four $S_a(T_1,5\%)$ levels (after Fragiadakis et al., 2006)

During the course of this qualitative review, it is observed that some researchers have described procedures to obtain mass, stiffness and strength ratios used in their studies, while some are silent, and very few have given numerical values of such ratios. A tabular summary of the types (mass, stiffness and strength) and the extent (mass ratios, stiffness ratios, and strength ratios) of vertical irregularities studied by various researchers is presented in Table 3. Here, mass ratio is defined as the ratio of mass of the story under consideration in the irregular structure to that in the regular structure. Stiffness and strength ratios are also defined in a similar manner. A regular structure is considered as one without any major discontinuities in mass, stiffness, and strength over its height.

CONCLUSIONS

From the above discussion, it can be concluded that a large number of research studies and building codes have addressed the issue of effects of vertical irregularities. Building codes provide criteria to classify the vertically irregular structures and suggest elastic time history analysis or elastic response spectrum analysis to obtain the design lateral force distribution. A majority of studies have evaluated the elastic response only. Most of the studies have focused on investigating two types of irregularities: those in set-back and soft and/or weak first story structures. Conflicting conclusions have been found for the set-back structures; most of the studies, however, agree on the increase in drift demand for the tower portion of the set-back structures. For the soft and weak first story structures, increase in seismic demand has been observed as compared to the regular structures. For buildings with discontinuous distributions in mass, stiffness, and strength (independently or in combination), the effect of strength irregularity has been found to be larger than the effect of stiffness irregularity, and the effect of combined-stiffness-and-strength irregularity has been found to be the largest. It has been found that the seismic behavior is influenced by the type of model (i.e., beam hinge model or column hinge model) used in the study. Finally, buildings with a wide range of vertical irregularities that were designed specifically for code-based limits on drift, strength and ductility, have exhibited reasonable performances, even though the design forces were obtained from the ELF (seismic coefficient) procedures.

Table 3: Summary of Types and Extent of Vertical Irregularities Studied by Various Researchers

Serial No.	Reference	Type of Building	Extent of Vertical Irregularities			Remarks
			Mass Ratio	Stiffness Ratio	Strength Ratio	
1	Ruiz and Diederich (1989)	5-Story (with Brittle Infill Walls)	-	4.0	0.65-2.0	Mass irregularities have not been studied and infill walls have been provided at all floors except at the first floor
		12-Story (with Brittle Infill Walls)	-	0.9	1.0-2.0	
		5-Story (with Ductile Infill Walls)	-	4.0	1.0-2.0	
2	Valmudsson and Nau (1997)	5-Story	0.1, 0.5, 1.5, 2.0, 5.0	0.5, 0.6, 0.7, 0.8, 0.9	0.5, 0.6, 0.7, 0.8, 0.9	
		10-Story	0.1, 0.5, 1.5, 2.0, 5.0			
		20-Story	0.1, 0.5, 1.5, 5.0			
3	Al-Ali and Krawinkler (1998)	10-Story	0.25, 0.5, 2.0, 4.0	0.1, 0.25, 0.5, 2.0, 4.0, 10.0	0.5	
4	Das and Nau (2003)	5-Story	2.5-5.0	0.09-1.6	0.27-1.05	
		10-Story	2.5-5.0	0.09-1.71	0.27-1.05	
		20-Story	2.5-5.0	0.08-1.89	0.27-1.07	
5	Chintanapakdee and Chopra (2004)	12-Story	-	0.2, 0.5, 2.0, 5.0	0.2, 0.5, 2.0, 5.0	Mass irregularities have not been studied
6	Fragiadakis et al. (2006)	9-Story	-	0.5, 2.0	0.5, 2.0	Mass irregularities have not been studied

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