16th World Conference on Earthquake, 16WCEE 2017

Santiago Chile, January 9th to 13th 2017

Paper N° 162

Registration Code: S-Q1461965905

IMPACTS OF VERTICAL IRREGULARITY ON THE SEISMIC DESIGN OF HIGH-RISE BUILDINGS

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Abstract

Many tall buildings are practically irregular, as a perfect regular high-rise building rarely exists. The structural irregularity increases the uncertainty related to the capacity of the building to meet the design objectives. There is a pressing need to systematically assess the impacts of vertical irregularity on the seismic design of tall buildings, particularly the extreme irregularity types. This study is thus devoted to evaluate the seismic design coefficients of the modern tall buildings with different vertical irregularity features. A brief survey of the most common vertical irregularities in reinforced concrete multi-story buildings is conducted to select reference structures. Five 50-story buildings are then selected and fully designed using three-dimensional finite element models and international building codes to represent well-designed regular and irregular tall buildings in Dubai, UAE, a medium seismicity region that is selected as a case study. Fiber-based simulation models are developed to assess the seismic response of the five benchmark buildings under the effect of forty earthquake records representing far-field and near-source seismic scenarios. The selected real earthquake records account for the ground motion uncertainty and the seismicity of the case study area. The comprehensive results obtained from a large number of inelastic pushover and incremental dynamic analyses provide insights into the local and global seismic response of the reference structures and confirm the unsatisfactory response of tall buildings with severe vertical irregularities. Due to the significant impacts of the severe irregularity types on the seismic response of tall buildings, the conservative code coefficients are recommended in design. The study also concluded that although the design coefficients of regular tall structures and buildings with insignificant irregularities are adequately conservative, they can be revised to arrive at a more cost-effective design of tall buildings.

Keywords: vertical irregularity; tall buildings; seismic design codes; dynamic response; seismic design coefficients

1. Introduction

The structural irregularity is widely observed in buildings as a result of the architectural and service requirements in the design process, errors and modifications during the construction phase, and changes in the building use throughout its service life. Modern seismic design codes distinguish between the plan and vertical irregularity [1, 2]. The plan (horizontal) irregularity occurs as a result of several reasons such as when the structure is significantly subjected to torsion or exhibit a discontinuity in the lateral force resisting system (LFRS) out of its plane (out of plane offset). The vertical irregularity may occur due to significant changes in the stiffness, strength, mass, dimensions, or a discontinuity in the LFRS plane. The tendency to distinguish between irregularity in plan and in elevation also characterizes the scientific literature. The growing interest in investigating the seismic behavior of building irregularity has been shown in the literature, particularly for vertical irregularity [3-5]. However, the impacts of different types of vertical irregularity on the seismic design of buildings have not been systematically covered in the literature, particularly the extreme irregularity of real-life high-rise structures.

The definitions of international codes for different types of vertical irregularity is summarized in Table 1. It is shown that international design codes basically categorize the vertical irregularity to five types: (i) stiffness, (ii) mass, (iii) geometry, (iv) in-plane discontinuity of vertical force resisting element, and (v) discontinuity in the LFRS strength. Unlike Eurocode-8 [1], ASCE/SEI-7 [2] divides the stiffness irregularity into: (i) soft story, and (ii) extreme soft story; while the discontinuity of lateral strength is divided into: (i) weak story, and (ii) extreme weak story. These detailed definitions of the soft story and weak story irregularities reflect the significance of imposing different requirements according to the level of severity of these two types of irregularities.

Table 1 – Definition of vertical irregularity according to international seismic design codes [1, 2]

Type of vertical irregularity	Design code: U.S. design standards and guidelines [2, 6]	Design code: European design code [1]
Stiffness - soft story	$K_i < 70\% K_{i+1}$	$K_i \neq K_{i+1}$
Stiffness - extreme soft story*	$K_i < 60\% K_{i+1}$	N/A
Mass	$M_i > 150\% M_{i\pm1}$	$M_i \neq M_{i+1}$
Geometry*	$L_i > 130\% L_{i\pm1}$	$L_i > 120\text{-}150\% L_{i+1}$
In-plane Discontinuity*	$L_o > L_b$	when discontinuity exists
Discontinuity in lateral strength - weak story	$Str_i < 80\% Str_{i+1}$	$Str_i \neq Str_{i\pm1}$
Discontinuity in lateral strength - extreme weak story*	$Str_i < 65\% Str_{i+1}$	N/A

K_i: Stiffness of the soft story; K_{i+1}: Stiffness of the floor above the soft story
M_i: Mass of a story; $M_{i\pm1}$: Mass of adjacent story
L_i: Length of a story; $L_{i\pm1}$ or L_{i+1}: Length of the story adjacent to or above the irregular one
L_o: Vertical element offset; L_b: Vertical element length in the story below the irregular story
Str_i: Lateral strength of weak story; $Str_{i\pm1}$ or Str_{i+1}: Lateral strength of adjacent story or the story above the weak one
$\neq$: Indicates a significant change; *: Irregularity types investigated in the present study

An analytical model of a structure that accounts for all sources of stiffness, P-delta effects, and the inelastic response is the most accurate approach for the seismic design. Development of such an analytical model is costly, and hence the inelastic seismic response is accounted for in modern design approaches through the use of the response modification factor, R, deflection amplification factor, C_d, and overstrength factor, Ω_o [1, 2, 7]. These factors are termed the seismic design response factors in this study. FEMA-P695 [7] proposed an approach to quantify these design factors, in which the R factor was related to the ratio of the spectral acceleration of the maximum considered earthquake at the period of the structural system, S_{MT}, and the seismic response coefficient, C_s. The Ω_o factor was related to the ratio of the ultimate strength of the structure, S_{max}, to the C_s coefficient. The R and C_d factors were considered to be equal in this approach. The results of incremental dynamic analysis (IDA) and inelastic pushover analysis (IPA) were utilized to evaluate the seismic design factors in other previous studies [8-10]. The R factor is calculated as follows: $R=(PGAc/y)\ \Omega_{fy}$, where PGAc/y is the ratio of the peak ground acceleration (PGA) at collapse to the PGA at the first indication of yielding, and Ω_{fy} is the overstrength factor at the first indication of yielding. The deflection amplification factor was considered to be equal to IDRc/y, which is the ratio of the maximum interstory drift ratio (IDR) corresponding to the collapse prevention limit state to the IDR at the first indication of yielding. The calculated seismic design factors in the above-mentioned studies were compared with the code values, which proved that the latter coefficients were conservative for regular structures.

The above-mentioned brief review highlights the pressing need to systematically assess the seismic design approach and coefficients recommended by building codes for different types of irregular high-rise buildings and to verify their relative safety margins. The main objectives of the current study are thus twofold: (i) to select, design and idealize a diverse range of vertically irregular buildings to represent real-life high-rise structures; and (ii) to assess the impacts of different types of vertical irregularity on seismic design using a wide range of input ground motions and rational performance criteria aiming at providing practical recommendations for the design of this important class of buildings.

2. Selection and Design of Vertically Irregular Structures

One of the main tasks of the current study is to select representative vertically irregular buildings. The selected reference buildings are selected based on a concise survey of the common types of irregular high-rise structures in the UAE, a medium seismicity region that is selected as a case study due to its rapid rate of high-rise building construction. Abrupt changes in the stiffness, geometric dimensions, and/or strength of the LFRS along the building height due to architectural and services requirements represent the most common vertical irregularities in the case study area. Five 50-story reinforced concrete (RC) high-rise buildings are thus selected for the purpose of the current study. The selected buildings are denoted B1-REG, B2-SST, B3-GEO, B4-DIS and B5-WST, which characterize a regular structure, extreme soft story irregularity, geometric irregularity, in-plane discontinuity irregularity, and extreme weak story irregularity, respectively, as shown in Table 2. The regular building is used for comparison with other irregular structures. It is noteworthy that generating a certain type of irregularity may result in some stiffness changes. For instance, the introduced vertical irregularities at the lower stories of buildings B3-GEO, B4-DIS and B5-WST have an influence on the stiffness distribution. However, these stiffness changes do not lead to stiffness - soft story/extreme soft story irregularity [2]. The definitions of the selected building irregularities are as per the ASCE-7 provisions [2], as explained in Table 1. Table 2 summarizes the main structural characteristics of the selected reference structures while Fig. 1 depicts their layouts, configurations and LFRSs.

Table 2 – Characteristics of benchmark irregular building

Building reference	**Building irregularity type**	**Typical story height (m)**	**Ground story height (m)**	**First basement height (m)**	**Total height (m)**
B1-REG	Regular building	3.2	3.2	3.2	160
B2-SST	Stiffness/ extreme soft story irregularity	3.2	6.5	3.2	163.2
B3-GEO	Geometric irregularity	3.2	3.2	3.2	160
B4-DIS	In-Plane Discontinuity irregularity	3.2	4.7	4.7	163
B5-WST	Discontinuity in lateral strength/weak story irregularity	3.2	3.2	3.2	160

Shear walls are mainly used as the lateral force resisting elements since they are efficient in controlling the lateral deformations developed by wind or earthquake loads. Flat slabs with marginal beams are employed as horizontal diaphragms to transfer the gravity loads to vertical elements. In buildings B4-DIS and B5-WST, columns are used at the lower stories as a result of their irregularities, as shown in Fig. 1(c-d). The five reference structures are fully designed for the purpose of this study. Three-dimensional (3D) simulation models are developed using ETABS [11], which is widely used for the design of the multi-story buildings. Modal response spectrum analysis (MRSA) is employed to estimate the lateral seismic forces. The 3D ETABS models account for the stiffness and strength values of structural members as per the design code [12]. The concrete strength, f_c`, varies throughout the height of vertical element starting from 48 MPa (cube strength, f_{cu}, of 60 MPa) at the foundation to 32 MPa (f_{cu} of 40 MPa) at the roof. Cube concrete strength of 40 MPa is used for all slabs and beams. The yield strength, f_y, of reinforcing steel bars is 460 MPa for flexural design and 420 MPa for shear design [12]. Permanent loads include the self-weight of structural members with superimposed dead load of 4.0 kN/m^2. Live loads are 2.0, 4.8 and 3.0 kN/m^2 for the residential areas, corridors and staircases, and basements (parking areas), respectively [2]. The case study area (Dubai, UAE) represents a region of medium seismicity. The spectral response accelerations at 0.2 sec and 1.0 sec are 0.83 g and 0.24 g, respectively, while the site class is 'C' (very dense soil). The R factor is 4.0, Ω_o factor is 2.5, and seismic design category, SDC, is 'C' [2].

As per the design code, the in-plane discontinuity of LFRS and the discontinuity in lateral strength of LFRS (weak story irregularity) should be designed using special cases of loading [2]. Both ultimate limit state and serviceability load combinations are considered in the design process. Service load combinations are employed to verify the vertical and lateral deformations while the structural elements are designed using the ultimate load combinations [2, 12]. The ultimate load combinations, including the vertical and horizontal seismic load effects, are as follows: $(1.2+0.2S_{DS})\ D + \rho\ Q_E + L$ and $(0.9-0.2S_{DS})\ D + \rho\ Q_E$, where S_{DS} is the design spectral response acceleration at 0.2 sec, ρ is the redundancy factor, D is the dead load, and Q_E is the horizontal seismic force. The above combinations are adopted for the design of the regular structure as well as the buildings with stiffness and geometric irregularities. The Ω_o factor is utilized for the design of the irregularity introduced in buildings B4-DIS and B5-WSST [2]. Table 3 shows a comparison between the periods of vibration of the five reference buildings obtained from the finite element (FE) models used in design as well as the fiber-based (FB) models developed for seismic assessment, as subsequently discussed. With the exception of building B3-GEO, which has slightly shorter periods, the fundamental periods of the irregular buildings in the transverse direction are longer than the regular counterpart (B1-REG). This is attributed to the reduced stiffness of the lower stories in buildings B2-SST, B4-DIS and B5-WST. On the other hand, the footprint of the lower stories of building B3-GEO is larger than those of the regular structure, and hence the stiffness increases and period decreases.

Straining actions generated by gravity and lateral loads are considered in the slab design. The design of vertical elements (columns, shear walls, and core walls) is fully automated using ETABS. Although the boundary elements of shear walls and cores walls are not required by the design code for SDC "C" [12], they are utilized in design to enhance the seismic performance. The comprehensive design results of the five reference structures including the reinforcement details of shear walls and floor slabs are discussed in more detail by Khalifa [13]. The design results are used to develop the fiber-based models used in the inelastic analysis, as discussed hereafter.

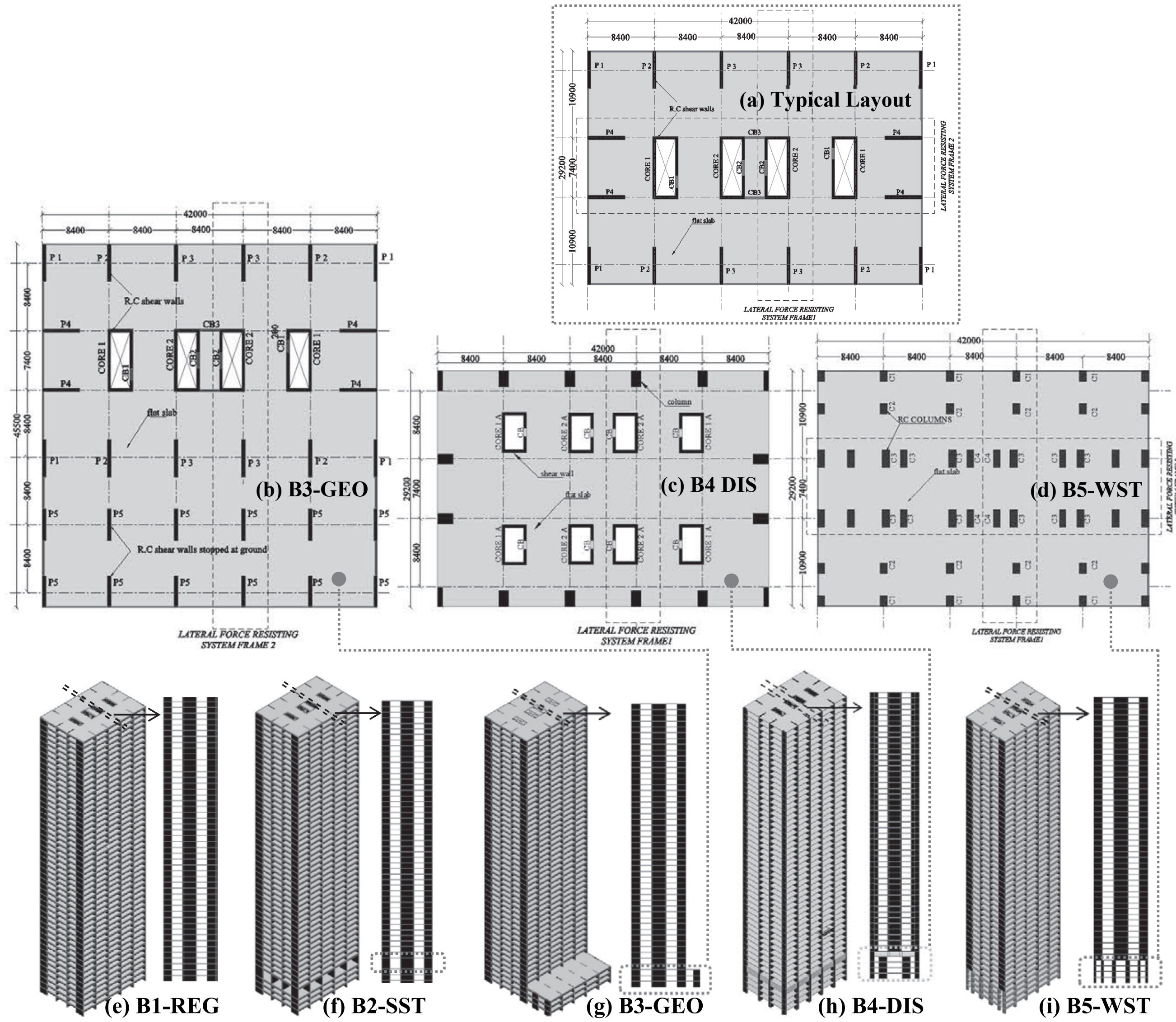

Fig. 1 – Description of reference structures: (a) typical layout of structures except at the irregularity levels; (b-d) layouts of B3-GEO, B4-DIS and B5-WST at the irregularity levels, respectively; and (e-i) configuration and LFRSs of reference structures

Table 3– Comparison of the uncracked periods of vibration for the reference buildings obtained from finite element and fiber-based models in the transverse directions

Modeling approach		B1-REG	B2-SST	B3-GEO	B4-DIS	B5-WST
Finite element (FE) models	T1	4.688	4.822	4.603	4.815	5.021
	T2	1.326	1.368	1.300	1.317	1.431
Fiber-based (FB) models	T1	4.540	4.673	4.280	4.561	4.835
	T2	1.220	1.240	1.204	1.230	1.300

3. Modeling of Irregular Buildings and Selection of Earthquake Records

Structures behave in a nonlinear manner during strong earthquakes, and hence the seismic assessment of buildings should be performed using inelastic dynamic time-history analysis (THA) [e.g. 6]. The nonlinear platform Zeus-NL is employed in the current study to conduct a large number of inelastic analyses [14]. Several verifications have been conducted for this analysis platform against full-scale tests carried out in Europe and the U.S. [15, 16]. For instance, comparisons of the full-scale test results for a three-story RC irregular building with those obtained from Zeus-NL confirmed the rational prediction of this analysis platform [15]. Moreover, the Zeus-NL modeling approach and key modeling parameters adopted in the present study were verified by comparisons with the nonlinear dynamic response of a full-scale seven-story wall building slice tested at UCSD [16]. Large research projects covering multi-span bridges and high-rise buildings were also conducted using the adopted analysis platform [e.g. 10, 17]. It is assumed in the present study that each reference building consists of four comparable LFRSs in the transverse direction, as depicted in Fig. 2. Simulation models are developed for the five reference buildings using Zeus-NL to represent the LFRSs in the transverse direction. Each of the idealized framing systems resists the lateral seismic forces in addition to gravity loads, including 25% of the total mass of the building. It is assumed that the exterior structural members only support gravity load, and hence the seismic forces are entirely resisted by the internal LFRSs. In the longitudinal direction, only one frame resists the lateral loads and the whole building mass, while other structural members only support gravity loads. It is noted that the transverse direction of the reference buildings is more vulnerable than the longitudinal direction. Additionally, certain irregularities are introduced in the transverse direction, and hence the nonlinear analysis is conducted in this direction.

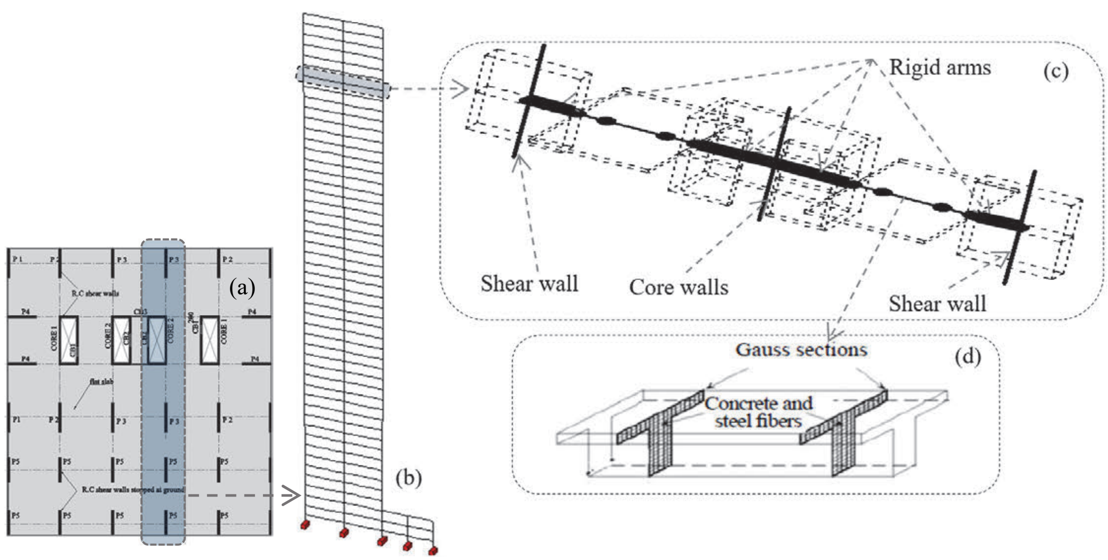

Fig. 2 – Modeling approach of reference structures for inelastic analysis: (a) B3-GEO layout; (b) Zeus-NL model; (c) geometrical modeling of horizontal and vertical elements; and (d) fiber based modeling

RC flexural wall, hollow rectangular, rectangular and T-sections are used to idealize shear walls, core walls, columns and slabs, respectively. The rigid arm length is the distance between the centerline and the face of the vertical elements, as presented in Fig. 2(c). Three cubic elastoplastic frame elements are used to idealize each horizontal and vertical structural member (slabs, columns, shear walls and core walls). This allows utilizing different cross-sections for each structural member, one at each member ends and another one at the mid-span. These sections help to accurately model different reinforcement profiles of structural members according to the design. Fig. 2(d) shows the Zeus-NL cubic elastoplastic element, which includes two Gauss section, as well as the concrete and reinforcing steel fibers. This modeling approach effectively represents the spread of inelasticity within the cross-section and along the member length. Reinforcing steel, confined concrete and unconfined concrete are idealized using this fiber modeling approach.

A uniaxial constant confinement concrete model and a bilinear elastoplastic steel model with kinematic strain-hardening are used in the Zeus-NL models [14]. Fully confined concrete is used in columns and in the boundary elements of shear walls and core walls. Partially confined concrete is used in the web of shear walls and core walls while unconfined concrete is used to model the concrete cover. The expected material strength is used to assess the seismic response of the reference structures [18]. Hysteretic damping is accounted for in the elastoplastic fiber element used to model structural members. The non-hysteretic damping is caused by many sources such as the nonstructural components. The latter type of damping is considered by utilizing stiffness-based Rayleigh damping [19]. The stiffness proportional damping is calculated for each reference building using the equivalent period of each structure as proposed by Alwaeli et al. [16]. Eigenvalue analysis is used to verify the vibration periods and deformed shapes of the reference buildings. The comparison shown in Table 2 for the uncracked periods of vibration obtained from the 3D ETABS models used in the design with those obtained from the Zeus-NL FB models shows the minor reduction in the period of vibrations obtained from the latter models. This reduction is due to the effective modeling of rebar in Zeus-NL, which increases the stiffness of structural elements unlike the ETABS models. The above-mentioned results and discussion validate the developed Zeus-NL models for the assessment of the seismic design response factors of the reference buildings using IPA and IDA.

The selection of input ground motions for the seismic assessment of high-rise buildings is a critical task due to the wide range of vibration periods of significance. Several previous studies concluded that the seismological parameters such as the earthquake magnitude and distance have significant effects on the dynamic analysis results [20]. In the current study, the seismological and site parameters, including the record magnitude, epicentral distance, soil class, ratio of PGA to peak ground velocity (a/v), and PGA are considered in the selection of earthquake records to represent the seismic scenarios expected in the case study region (Dubai, UAE). Three approaches for seismic performance assessment of buildings are recommended by NEHRP [21]; intensity-based assessment, scenario-based assessment, and risk-based assessment. The selection of the seismic records depends on the implemented type of assessment. In the current study, a scenario-based assessment is implemented as per the recommendation of a number previous studies for the case study region [e.g. 20]. The employed seismic scenarios represent: (i) severe events with a long epicentral distance, and (ii) moderate earthquakes with a short distance from the epicenter. For far-field events, a magnitude (M_w) range of 6.93 to 7.64, epicentral distance range of 91 to 161 km, stiff and very dense soil classes, low a/v ratio (<0·8 g/m s−1), and a PGA range of 0.9 to 2.39 m/s^2 are considered in the record selection. Furthermore, for the near-field records, a magnitude range of 5.14 to 6.04, epicentral distance range of 2.86 to 29.9 km, stiff and very dense soil classes, high a/v ratio (>1·2 g/m s−1), and a PGA range of 0.85 to 4.96 m/s^2 are considered in the selection of earthquake records. Two databases are used to select the input ground motions, which include the Pacific Earthquake Engineering Research center database [22] and the internet site for European Strong-motion Database [23]. From the selected databases, 20 far-field and 20 near-field natural records are selected to represent the earthquake scenarios in the study region. The far-field records fit the design response spectrum in the long period range, while the near-field seismic events match the design spectrum in the short period range, as shown Fig. 3 [2]. The above-mentioned two seismic scenarios account for the uncertainty of input ground motions. The selected records are scaled to a PGA of 0.16g before applying to the reference building models, which represents the design PGA for 10% probability of exceedance in 50 years [20].

4. Performance Limit States

The local and global inelastic response of the reference structures and the results of previous experimental studies are used in the current study to select the IDRs corresponding to the limit states needed to assess the seismic design response factors. For the local response, the IDR corresponding to the first indication of yielding in reinforcing steel represents the immediate occupancy (IO) limit state, while the first indication of crushing in the confined concrete of vertical structural elements corresponds to the collapse prevention (CP) limit state. Moreover, IDAs are carried out in order to define the global limit states from the IDA curves of the five reference buildings. Equivalent inelastic periods for the five reference structures are calculated and used to obtain the corresponding spectral acceleration of the selected input ground motions. The equivalent inelastic periods are calculated based on the first three inelastic periods weighted by the mass participation ratios [16]. The IDA results are used to develop the relationship between the maximum IDRs and spectral accelerations, as shown from the sample results presented in Fig. 4 for B3-GEO and B5-WST. The IO limit state is defined at the first deviation from the elastic response while the CP limit state is determined when the stiffness reaches 20% of the elastic value. The IDR corresponding to the IO and CP performance criteria are estimated at the 16 percentile of the lognormal distribution, as shown in Fig. 4. Following the above-mentioned approach, the IO limit states of B1-REG, B2-SST, B3-GEO, B4-DIS and B5-WST are 0.49%, 0.48%, 0.51%, 0.27%, and 0.44%, respectively. These values are consistent with those obtained from the local response results. For the B1-REG building, the IDR corresponding to the IO limit states (0.49%) is also consistent with the value recommended by design guidelines and previous studies [e.g. 24, 25].

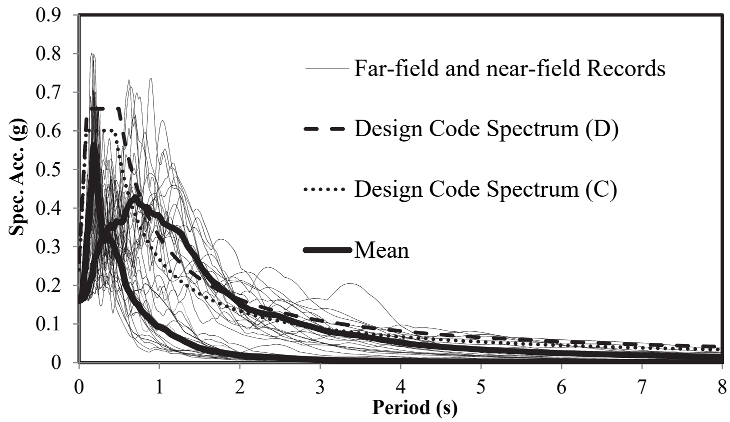

Fig. 3 – Response spectra of 40 earthquake records representing far-field and near-source events along with the design response spectra of site classes "C" and "D"

The IDR corresponding to the CP limit state of B1-REG, B2-SST, B3-GEO, B4-DIS and B5-WST are 4.97, 4.56, 6.08, 2.17 and 3.61, respectively. It is noted that the CP limit states calculated using the IDA curves are higher than those obtained from other approaches. The most conservative IDR corresponding to the CP limit state proposed in previous experimental studies related to shear wall structures (2.27%) is therefore adopted in the present study [24]. The selected CP limit state is slightly higher than that proposed by ASCE/SEI-41 [25]. Moreover, due to the insufficient experimental studies and the lack of code recommendations for the CP limit state of irregular high-rise buildings as well as the dispersion of the results observed from the IDA results, the most conservative CP limit states are adjusted using the CP value adopted for the regular building (i.e. 2.27). The aforementioned approach results in CP limit states of 2.26%, 2.39%, 1.18% and 1.38% for B2-SST, B3-GEO, B4-DIS and B5-WST, respectively. The comparable results obtained from IPA and IDA of B1-REG, B2-SST and B3-GEO, as subsequently discussed, lend weight to the CP values adopted for these three buildings. Moreover, it is important to note that the selected CP performance criteria of B4-DIS and B5-WST are conservative and in line with those recommended in few previous experimental studies related to these types of irregularities [26].

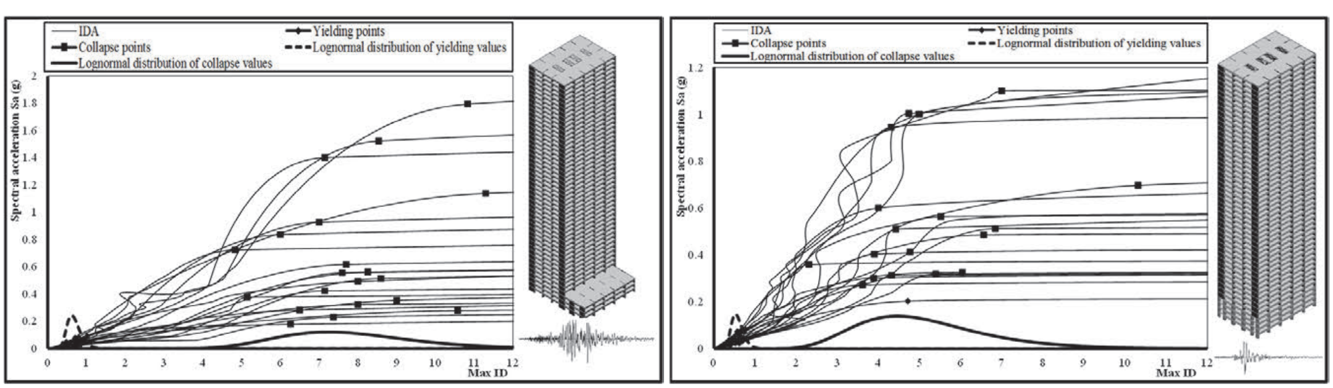

Fig. 4 – IDA curves obtained from twenty long-period earthquake records showing the first yielding and collapse points as well as their lognormal distributions: (a) B3-GEO; and (b) B5-WST

5. Impacts of Vertical Irregularity on Seismic Design

Although IDA is mainly employed to assess the seismic design response factors under a wide range of earthquake records, IPA is firstly conducted to obtain an initial estimation of the lateral capacity and to estimate the Ω_o factor of the five reference buildings. The developments of plastic hinges and crushing in concrete are also traced in structural members using IPA. A number of previous studies concluded that the IPA accuracy was not significantly depreciated even for irregular structures while other studies concluded that the uniform lateral load distribution can be used to conservatively estimate the initial stiffness and lateral capacity of high-rise buildings [e.g. 20]. In the current study, IPA with a lateral load pattern representing the mass distribution throughout the building height is deployed to: (i) compare between the capacity curves of the regular and irregular structures, and (ii) initially assess the local response of structural elements and map it with the adopted global damage measure.

A comparison between the capacity curves of B1-REG and B4-DIS is shown in Fig. 5(a). The IDRs corresponding to the first yielding and crushing in structural members as well as the global yielding are mapped on the capacity curves and are summarized for the five reference buildings in Fig. 5(b). The ultimate strength obtained from IPA represents a conservative estimate of the lateral capacity of a building, as previously discussed. The global yielding is evaluated from an elastoplastic idealization of the real capacity curve [9]. The idealize capacity envelopes of the reference buildings are depicted in Fig. 6. It is observed from the IPA results that the ultimate strength, initial stiffness and ductility (ultimate-to-yield displacement) of the building with the extreme soft story irregularity (B2-SST) are slightly lower than those of the regular structure (B1-REG). These minor differences between the response of B2-SST and B1-REG validate the design code approach regarding this type of irregularity (i.e. no special requirements are needed in the design to SDC 'C') [2]. In contrast, the above-mentioned characteristics (i.e. strength, stiffness and ductility) of B3-GEO are slightly higher than those of the B1-REG building due to the increase in the footprint of the former structure. Again, this observation validates the code approach towards this type of irregularity. For B4-DIS and B5-WST, the ultimate capacity and initial stiffness are much higher than those of B1-REG, while the ductility significantly decreases, as shown in Figs. 5(a) and 6. These differences in response are mainly due to the use of the Ω_o factor in the design of buildings B4-DIS and B5-WST at the irregularity levels as required by the design code [2]. Although the initial stiffness and ultimate strength of the latter two buildings are improved, the ductility reduction supports the code conservative approach toward the design of these types of vertical irregularity, particularly regarding the use of special load cases in design. The results presented in Fig. 5(b) also support the above-mentioned observations.

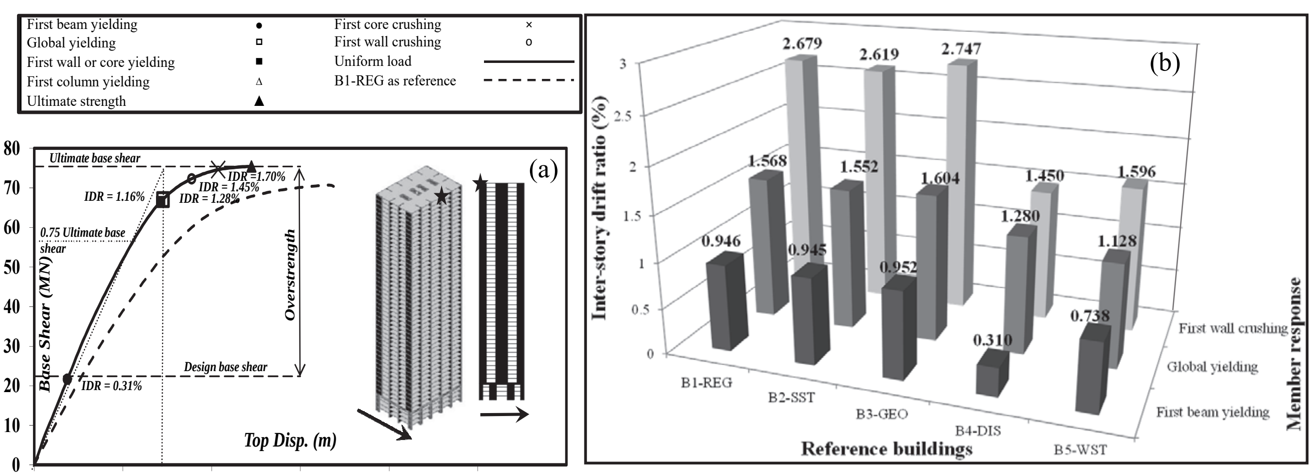

Fig. 5 – Comparisons of the IPA results of the irregular and regular structures: (a) B4-DIS vs B1-REG, and (b) IDRs at the first indication of member yielding and crushing

The actual strength is influenced by several parameters such as the design safety factors, material characteristics and structural system [9]. The structural overstrength is the actual-to-design strength, as shown in Fig. 5(a) and 6. The Ω_o factor is measured in the present study at different levels, including the first indication of yielding, global yielding and ultimate capacity. Fig. 7 depicts the overstrength factors at the first plastic hinge, Ω_{fy}, global yielding, Ω_{gy}, and ultimate capacity, Ω_u, of the reference buildings. It is shown that the overstrength factors of B1-REG and B2-SS buildings are comparable, which confirms the marginal effect of the soft story irregularity on lateral capacity. Due to the higher design base shear of building B3-GEO, the overstrength factors of this structure are lower than the regular one. The overstrength factor at the first plastic hinge of building B4-DIS is significantly lower than that of other buildings due to the early yielding of the transfer slab, which supports the heavy vertical load from typical stories (refer to Fig. 1). The overstrength factors of the latter building at global yielding and ultimate capacity are slightly lower than those of the regular building. The overstrength factors of the B5-WST building are higher than those of other structures due to the use of an Ω_o factor in design [2].

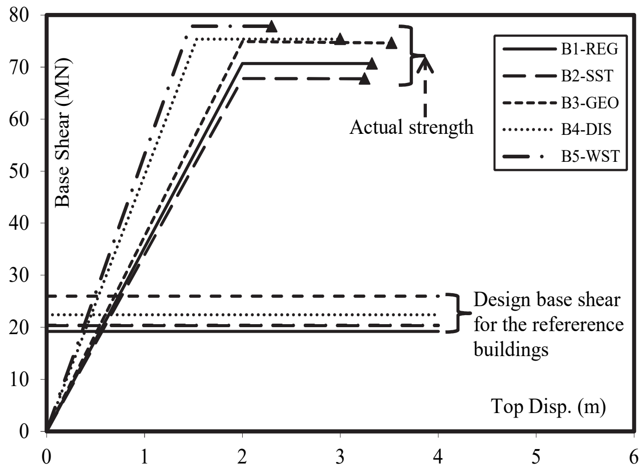

Fig. 6 – Idealize capacity envelopes of the reference buildings

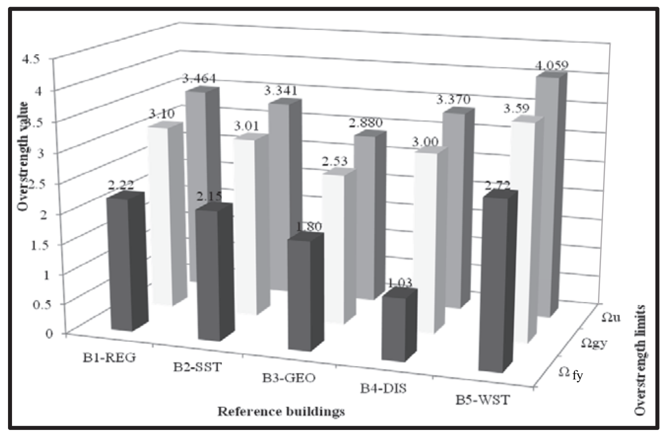

Fig. 7 – Overstrength factors of the reference buildings at the first indication of yielding, Ω_{fy}, global yielding, Ω_{gy}, and at ultimate capacity, Ω_u

The R and C_d factors of the regular and irregular benchmark structures are estimated following the approach previously discussed. The R factor is the collapse-to-yield PGA ratio, PGAc/y, times the Ω_{fy} factor while the C_d factor is considered equal to the collapse-to-yield IDR, IDRc/y [8, 10]. The IDA results using the selected 20 long period records are employed to evaluate the R and C_d factors as a result of their higher impact on the response of the reference structures [13]. The IDRs and PGAs ratios at yield and collapse for the five reference buildings are depicted in Figs. 8 and 9, while Figs. 8(f) and 9(f) depict the median values. It is shown that PGAc/y is larger than IDRc/y for the five reference structure. These differences reflect the margin of safety when C_d is considered equal to R, as per the design code [7].

The overstrength factors at the first indication of yielding for the five reference buildings are calculated using IPAs and IDAs, as shown in Fig. 10. Since the IDA results are more reliable than those of IPA for long period structures, particularly irregular buildings, the overstrength factors calculated using IDA are adopted to evaluate the seismic design factors. Fig. 11 depicts a comparison between the calculated R and C_d factors of the five benchmark buildings with the code recommended values. The R and C_d factors of the B2-SST and B3-GEO buildings are comparable to the regular structure (B1-REG). On the other hand, the R and C_d factors of the B4-DIS and B5-WST buildings are lower than those of B1-REG. This is attributed to the significant irregularity and the use of overstrength factor (Ω_o) in the design of the lower stories of the latter two buildings. It is shown from Fig. 11 that the code recommended factors are conservative for the five reference systems. The results of the present study confirm that the impacts of different irregularity types on the seismic design response factors vary. The discontinuity of the LFRS and the weak story irregularity, which are represented by B4-DIS and B5-WST, have the highest impact on the seismic design response factors. The results indicate that the R factors of the regular structure and buildings with insignificant irregularity (i.e. B2-SST and B3-GEO) can be initially increased by 10%. Further adjustment is possible after a careful response assessment of the structures designed using the suggested reduction in seismic design forces. Due to the significant impact of irregularity on the local and global response of buildings with a discontinuity in LFRS and weak story, the conservative R and C_d factors of the design code are recommended.

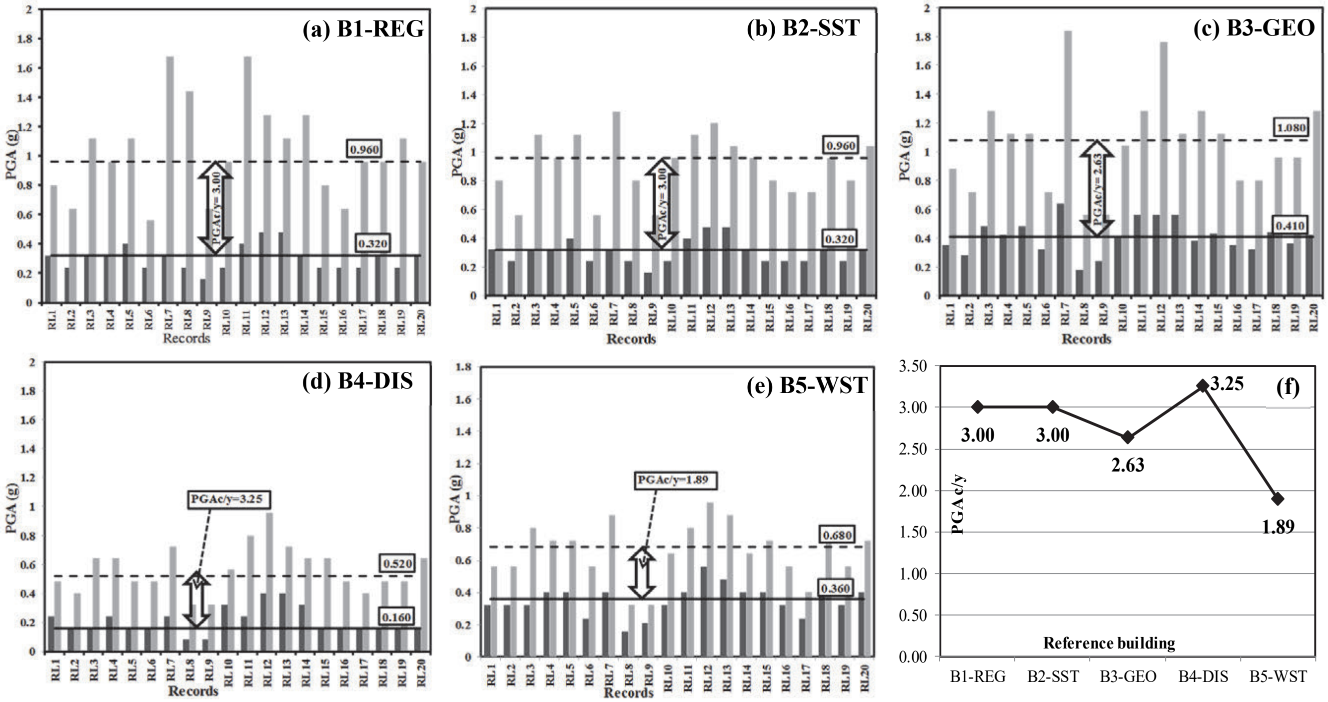

Fig. 8 – IDA results of five reference buildings using twenty long-period earthquake records: (a-e) collapse-to-yield PGA ratios, PGA(c/y); and (f) summary of the median PGA(c/y) ratios

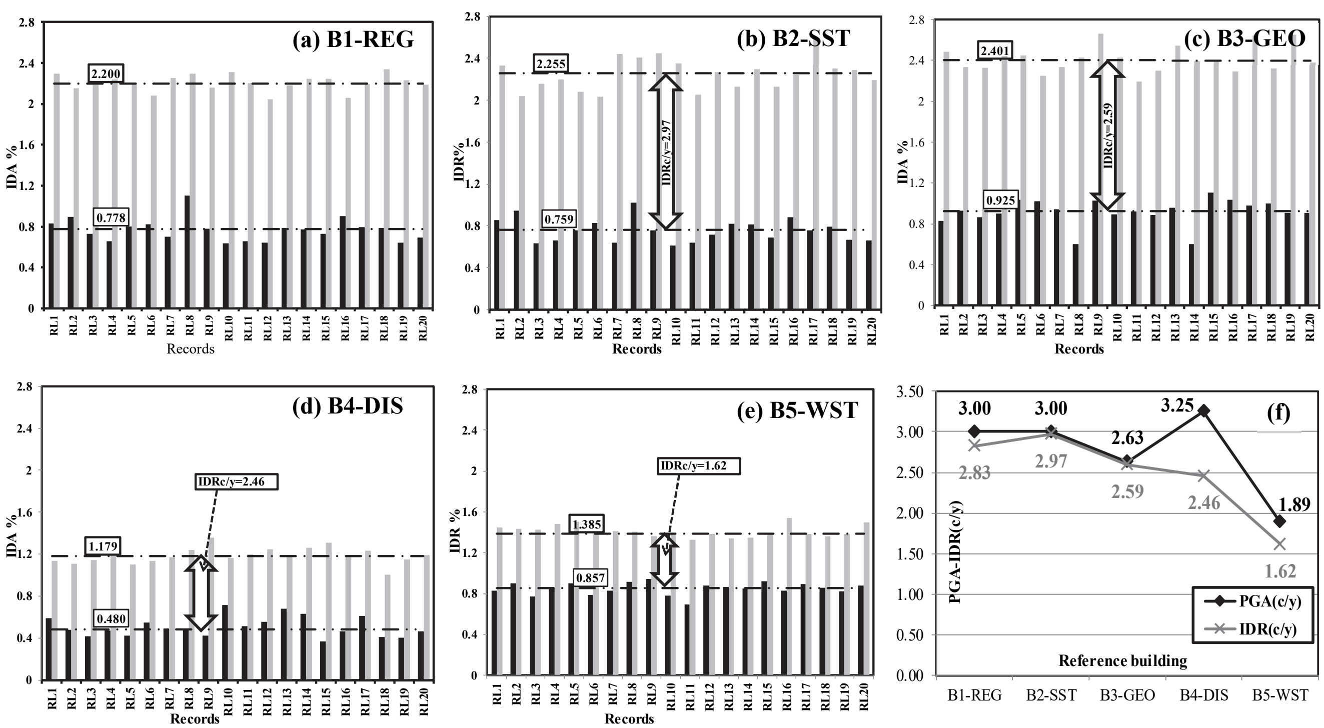

Fig. 9 – IDA results of five reference buildings using twenty long-period earthquake records: (a-e) collapse-to-yield IDRs, IDR(c/y); and (f) relationship between PGA(c/y) and IDR(c/y)

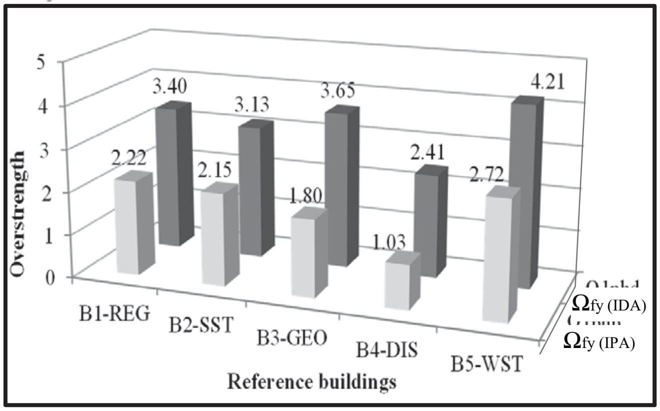

Fig. 10 –Overstrength factors obtained from IPA and IDA

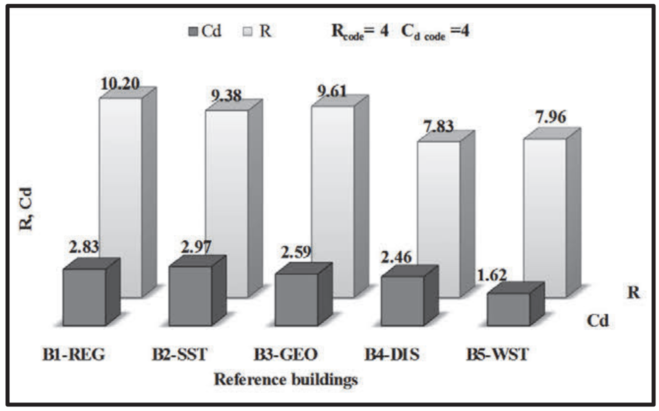

Fig. 11 – R and C_d factors of reference buildings using twenty long period input ground motions

6. Conclusions

The aim of this study was to assess the impacts of the most common vertical irregularity types on the design of high-rise buildings. Five 50-story RC buildings denoted B1-REG, B2-SST, B3-GEO, B4-DIS and B5-WST were selected to represent well-designed regular and irregular high-rise buildings in a medium seismicity region represented by Dubai, UAE. The five reference buildings were fully designed and detailed for the purpose of this study using 3D finite element models according to international building codes. Inelastic fiber-based simulation models were developed and verified for the five benchmark structures. Two earthquake scenarios, applicable to several seismic regions, were selected to represent the seismicity of the case study area: (i) far-field earthquakes with a medium-to-high magnitude and long distant from the epicenter, and (ii) near-field events with a low-to-medium magnitude and a short site-to-source distance. The ground motion uncertainty was accounted for using 40 earthquake records representing the above-mentioned two seismic scenarios. A large number of IDAs were performed to assess the seismic performance of the five reference structures at different performance levels. The performance limit states were selected based on the comprehensive inelastic analysis results and the values recommended in previous experimental studies related to shear wall structures and irregular buildings. The following conclusions were drawn based on the findings of this study:

- Although the seismic design code recommends the use of an overstrength factor for the building with a discontinuity in LFRS (B4-DIS) and the structure with extreme weak story (B5-WST), the design process of these irregular buildings confirmed the need for imposing reduction limits on the cross-sections and steel ratios of the stories above the irregularity levels to avoid any sudden changes in stiffness and strength.
- With the exception of the B4-DIS building, the calculated overstrength factors using IDAs were more than those recommended by the design code. The relatively less satisfactory safety margin of the B4-DIS building was confirmed from the observed low level of reserve strength.
- For the regular high-rise building and structures with insignificant irregularity (i.e. B2-SST and B3-GEO), the response modification factors could be safely increased. An initial increase in the R factor of 10% is proposed while a further adjustment is possible after a careful assessment of the structures designed using the suggested reduction in seismic design forces. Due to the significant impacts of the irregularities related to discontinuities in LFRS or weak story on the local and global seismic response of high-rise buildings, the conservative code design forces are recommended to be retained.
- The calculated deflection amplification factors for the reference regular and irregular structures were significantly lower than those recommended by the design code. The code C_d factor could be initially decreased by 10% for shear wall structures while a more reduction is possible after the assessment of the structures designed using the proposed reduction in deflection amplification factors.

While this study provided general insights into the seismic design approach of structures with different types of vertical irregularity, it also recommended revising specific design coefficients of buildings with less significant irregularities to arrive at more cost-effective designs of real-life high-rise buildings.

7. Acknowledgements

This work was supported by the United Arab Emirates University under research grants 31N227 and 31N132.

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