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ANALYTICAL STUDY ON COLLAPSE-RESISTING CAPACITY OF VERTICALLY IRREGULAR STEEL MOMENT FRAMES

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Abstract

The present study investigates the collapse potential of vertical irregular moment-frame system based on the performance based plastic design methodology (PBPD). The well-known method uses the input energy as well as the plastic energy capacity of a building to design the yielding members so that the favorable yield mechanisms reach.

Various low-rise to high-rise steel framing are considered as case study. Steel beam–column members of these case studies are proportioned by the plastic energy based method and by the current elastic design method. In order to evaluate the capability of the PBPD to collapse prevention, key structural performance parameters for detailed steel moment framings in terms of maximum/mean inter-story drift ratios, residual drift ratios, and plastic hinge rotations are computed by nonlinear history analysis and then results are compared to the acceptance criteria recommended by the TBI Guidelines as well as the methodology reported in FEMA P695.

The comparison show that Performance based plastic design methodology is able to meet collapse margin which is a highest favorable mechanism of the tall vertical geometric irregular building whereas the current code-specified requirements are not practically fully adequate to satisfy the expected seismic behavior of high irregular buildings specifically under the maximum considered earthquake hazard level.

In addition, according the controlling criteria reported in TBI, two steel frames that proportioned by the PBPD method, is subjected to a set of ground motions with incremental intensities from maximum considered earthquake hazard level to the early collapse level to estimate a safety margin against life-threatening collapse.

The results exhibit that structural performance for each ground motion favorably shows safe margin against collapse as the maximum IDRs obtained from each records do not exceed 4.5%. In other words, Structural acceptance criteria based on the requirements of TBI Guidelines, for MCE hazard level, in terms of maximum/mean IDRs, RDRs and plastic rotations as local parameters are reasonably satisfied.

Keywords: Performance Based Plastic Design; high-rise moment resisting setback frame; collapse assessments, the energy balance concept, vertical irregularity.

1. Introduction

Many novel architectural designs cause complexities and irregularities in building structures which need to be thoroughly addressed by structural professionals.

Intensive research in restricting inelastic damage in irregular buildings has been done, but there is no codified procure to ensure the greatest possible yield of displacement control members and also prevent uneven structural collapse due to concentration of elastic deformation near the weak area for certain structures.

The so-called 'setback' is referred to one of the most common type of vertical irregularities partially leading to elimination of the structural bending resistance and discontinuity of the load transfer in the lateral bearing system. Additionally, setbacks cause abrupt shrinking of adjacent story plans which have detrimental effect on the response to seismic loadings as a result of dissimilar mass distribution in these stories [1].

This extra damage has been a concern in seismic building codes as well as the subject of many researches to improving seismic performance of irregular frames. An experimental study on a six story irregular moment resisting frame which subjected to earthquake simulations of varying intensity in order to identify the seismic performance of setback structures have been performed. The study focused on the influence of vertical irregularities on seismic response of the building. They indicated that there are several uncertainties associated with the nature of setback that using both the conventional dynamic and conventional static design methods are not sufficient to identify the damage concentration around the setback area. A modification of present design criteria may be necessary so that complies with the actual behavior of the irregular frames to be able prevent unfavorable failure mechanism [2].

In another study, the effects of varying degrees of setbacks including strength and stiffness irregularity on dynamic response of multi-story framed were investigated by nonlinear response history analysis (NL-RHA) and the results were compared by modal pushover analysis (MPA). Vertical irregularities specifically stiffness irregularity significantly affects the distribution of inter story drift on the height as well as roof displacements. It was concluded that vertical irregularities significantly affect the drift demand in the upper stories which can influence the response of lower stories. Even though the use of MPA provided more accurate demands than using the current modal analysis, the seismic demands for vertical irregular frames should be determined by NL-RHA [3].

Modern design codes to overcome these problems impose some limitations upon uncommon building shapes, complex structural systems, and especially structural heights. These limitations unfortunately restrict recent trends in modern high-rise architectural designs to provide adequate sunlight and ventilation for the bottom stories. Recently, researchers put performance-based analytical methods in practice to reliably estimate the seismic performance of tall building structural systems with irregular shapes [4], irrespective of the restrictions placed by design codes in order to reach a compromise between structural engineers and architects.

AISC341-10 [5] requires capacity design procedures of force-controlled actions in beams, columns, and panel zones to assure that system can withstand these large inelastic deformations without premature brittle failure. Since both structural stiffness/strength degradation and destabilizing p-delta effects are responsible for lateral instability in MRFs owing to severe earthquakes, FEMA P695 [6] provides a technique to measure potential safety against collapse for buildings by considering these effects.

In recent years, researchers have outlined new plastic design approaches in conjunction with the energy-based method for seismic design of new buildings and have recommended these requirements instead of using current elastic procedures. The well-known energy-based design first proposed by

Housner [7] who introduced the parametric equation for total input energy dissipated through elastic as well as inelastic behavior. Leelataviwat [8] employed the proposed input energy equation to extract design base shear by equating the total input energy to work which need to push the structure up to a preselected target drift.

Unlike elastic design procedures which use reduced seismic demands according to response modification coefficient to ensure inelastic behaviors, in the proposed design base shear by Leelataviwat [9], the inelastic behavior directly account on the determination of design base shear with assuming preselected yield mechanism.

Many other study were performed to extract a modification of the energy-based plastic design procedure which are named Performance based Plastic Design (PBPD) to create a procedure to design the structural member sizes aim to identify the unfavorable mechanism and prevent excessive damage [10,11].

Performance based Plastic Design (PBPD) concepts assume a target drifts and probable yield mechanism as key structural design parameters. General framework in this technique is based on the implementation of energy concept method which earlier proposed by Houser [7]. The design base shear for the specific hazard levels are extracted by equating the assumed input energy propose by Houser with the work required by the structure to achieve a desirable yield mechanism. The work-energy equation can be considered equation (1):

$$\left(E_e + E_p\right) = \gamma\left(\frac{1}{2}MS_v^2\right) = \frac{1}{2}\gamma M\left(\frac{T}{2\pi}S_a g\right)^2 \tag{1}$$

At the above equation E_e and E_p are, the elastic and plastic components of the energy needed to push the structure up to the maximum drift, respectively; S_v is the design pseudo-spectral velocity; S_a is the pseudo spectral acceleration; T is the natural period; and M is the total mass of the system.

The parameter γ, are defined as the energy modification factor which its amount depends on the structural ductility factor (μ_s) and the ductility reduction factor (R_μ), and can be calculated using the below formula:

$$\gamma = \frac{2\mu_s - 1}{R_\mu^2} \tag{2}$$

Unlike the current elastic design approach which despite of various type of buildings irregularity, consider the same reduction factor value to accounts for inelastic behavior in the design base shear equation, the plastic energy based method take in to accounts the intended yield mechanism behavior for calculating the design base shear. As the design control requirements like as drift control are accounts into the base shear equation of the direct plastic method, analysis of complex structural will need to be iterated to proper design section to be reached. In order to extract design base shear, the work-energy equation proposed by equation (1) can be rewritten in the following form:

$$\frac{1}{2}\left(\frac{W}{g}\right)\left(\frac{T}{2\pi}\times\frac{V_y}{W}g\right)^2 + V_y\left(\sum_{i=1}^{N}\lambda_i h_i\right)\theta_p = \frac{1}{2}\gamma\left(\frac{W}{g}\right)\times\left(\frac{T}{2\pi}S_a g\right)^2 \tag{3}$$

The required design base shear coefficient V_y/W can be reached by the admissible solution of Equation (3) as the following equation:

$$\frac{V_y}{W} = \frac{-\alpha + \sqrt{\alpha^2 + 4\gamma S_a^2}}{2} \tag{4}$$

Where a dimensionless parameter (α) is given by

$$\alpha = \left(h^* \times \frac{\theta_p 8\pi^2}{T^2 g}\right) \tag{5}$$

At the above equation, θ_p is named the plastic component of the target drift ratio and h* are defined as the following form [12]:

$$h^* = \sum_{i=1}^{N} (\lambda_i h_i) \tag{6}$$

In the present study, the plastic design approach (PBPD) were used to investigates the collapse potential of various moment-frame setback which currently lacking a modified design criterion.

At the first step, the size of frame components comprising beams and columns are calculated using

Both elastic (current design codes) and plastic design procedure so that designed members meet the requirement design criteria are explained as follow. Seismic behavior of the case study frames is evaluated both according the performance-based acceptance criteria reported in FEMA P695 [6]. Nonlinear simulations include all sources of nonlinearities in terms of stiffness and strength deterioration as well as destabilizing p-delta effects related to the seismic mass. Additionally, input ground motion sets representative of site-specific seismic hazard characteristics were fully determined.

In addition to obtaining code-specified parameters such as deflection amplification factor, collapse margin which is required for performance evaluation of the building were achieved. The obtained results by the two design methods were compared and results showed that the PBDP are more capable to predict the unfavorable mechanism in setback area which can eventually yield to entire collapse.

Finally, fragility functions as a tool to estimate probability of collapse for irregular steel MRFs were obtained by using wavelet-based damage sensitive features according in Ref. [13].

The elastic design procedure is based on ASCE7-10[14] as minimum requirements for structural loadings and AISC360-10 [15] and AISC 341-10[5] for design proportioning and seismic detailing, respectively. Key structural components such as beams; columns are designed based on the building located in regions with high seismicity. Moreover, other requirements such as strong column weak beam are satisfied during design process.

Equation (4) for V_y was determined with the assumption that plastic hinges will form at the beams before any collapse mechanism specifically around the setback area happen and also columns were considered as non-yielding members. In the following section, design parameters for both described design method are presented.

2. Case study definition and design parameters

The case studies are comprised of various irregular frames with different story from 10 to 50 stories to account the frame height on the seismic performance of the frames. All frames have a typical story height of 4 meter, while 3 basement stories have level height of 6 meter. Indeed, two main type of irregularity including geometric irregularity and stiffness irregularity were considered. The stiffness irregularity by defining structural walls through first five stories of high rise frames was modified.

The frames were considered as a special steel MRFs and beams and columns of the original frames were proportioned with I-shape and H-shape built-up sections using elastic design method in accordance with Load Resistance Factor Design (LRFD) of ANSI/AISC360-10 [15]. The frames were redesigned by the plastic design methodology as described earlier in section 4. A brief description of design parameters for 50 stories frames is presented in Table 1.

Table 1.Seismic demands and requirements for elastic and plastic design based on ASCE7-10 and AISC341-10

Parameter	Value
Site class (ASCE7-10, Table 20.3-1)	Class D
Response modification factor (ASCE7-10, Table 12.2-1-C.1)	$R = 8$
Over strength factor (ASCE7-10, Table 12.2-1-C.1)	$\Omega_0 = 3$
Deflection amplification factor (ASCE7-10, Table 12.2-1-C.1)	$C_d = 5.5$
Importance factor (ASCE7-10, Section 11.5.1)	$I_e = 1$
Spectral response acceleration parameter at short period	$S_{ds} = 0.95$
Yield drift ratio θ_y	1%
Target drift ratio θ_u	3%
Inelastic drift ratio $\theta_p = \theta_u - \theta_y$	2%
α	2.534
γ	0.685
R_μ	3

2.1. Materials

ASTM-A36, Grade 36 and ASTM-A572, Grade 50 steel is used for the beams and columns in the building, respectively. It is assumed that the nominal yield stress is 248.21 Mpa and 344.73 Mpa for A36 and A572, respectively. Additionally, the nominal ultimate stress is 399.90 Mpa and 448.15 Mpa for A36 and A572, respectively. The compressive strength of concrete is set to 30 Mpa for structural walls and floor slabs.

3. Description of the analytical model for NLRHA

The analytical model for nonlinear dynamic analyses is simulated using computer software CSI-SAP2000 [16], a general purpose finite element program. Nonlinear dynamic analyses for both original frames and PBPD frames are conducted according to the incremental intensity procedure of a collection of spectral-matched ground motions to achieve collapse intensities with different probability of occurrence.

Fixed-based boundary conditions are supposed for columns of the frames. P-delta effects are reflected in the model by applying gravity loads. Load combinations for p-delta effects are considered by 1.05 times dead loads plus 0.25 live loads according to FEMA P695[6]. Fiber-hinge elements are employed to capture flexural hinging in beams and axial-flexural hinging in columns and an inelastic joint model. Similar to nonlinear modeling of steel beams and columns, fiber models based on layered shell concept are employed to model concrete walls in which wall sections are subdivided into reasonable steel and concrete fibers.

The design-based spectrum (DBE) for RSA technique is obtained from the site-specific hazard investigation for 475-year return period (10% probability of exceedance in 50 year).Plastic hinging in beams, columns, and panel zones are considered by recommendations in ATC72 [17] and FEMA P440A [18] for monotonic and hysteretic behavior.

4. Ground motion records sets

Ground motions for nonlinear response history analysis (NLRHA) are selected from PEER NGA record [19]. These records, as stated in Table 2, include categories from moderate earthquakes (Mw=6.5) to very large earthquakes (Mw=7.9). As a consequence, inherent variability of ground motion features at structure site is taken into account. Ground motion record sets included in FEMA P695 are appropriate for buildings with natural periods less than or equal to 4 seconds. Numerous requirements for selecting and scaling ground motions, different from those reported in FEMA P695, in terms of controlling seismic hazard conditions, compatibility with the site conditions, and modification to match with the target spectrum are taken into account (Ref. [1]). Spectral-matching is utilized to adjust frequency contents of accelerograms in which the response spectrum is within predefined limits of a MCE design spectrum, site-specific spectrum, over the defined period band. Thus, the average of the square root of the sum of the squares (SRSS) 5% damped spectrum of all horizontal acceleration history pairs are approximately matched over period range 0.2T to 1.5T, where T is the fundamental period of vibration [1]. Spectrum-matched procedure is recommended by TBI for tall buildings, which tends to reduce dispersion of response values compared to results obtained from amplitude-matched procedure [4].

Table2.Selected ground motion records

Record Seq. No.	Event	Year	Station	Magnitude (M_w)	Mechanism	R_{rup} (km)	$V_s(30)$ (cm/s)
RSN143	Tabas, Iran	1978	Tabas	7.4	Reverse	2.05	767
RSN182	Imperial Valley-06	1979	El Centro Array #7	6.5	Strike Slip	0.56	211
RSN802	Loma Prieta	1989	Saratoga - Aloha Ave	6.9	Reverse Oblique	8.50	381
RSN879	Landers	1992	Lucerne	7.3	Strike Slip	2.19	1369
RSN1114	Kobe, Japan	1995	Port Island	6.9	Strike Slip	3.31	198
RSN1176	Kocaeli, Turkey	1999	Yarmica	7.5	Strike Slip	4.83	297
RSN1501	Chi-Chi Taiwan	1999	TCU063	7.6	Reverse Oblique	9.78	476
RSN1602	Duzce, Turkey	1999	Bolu	7.1	Strike Slip	12.04	294
RSN2114	Denali, Alaska	2002	TAPS, Pump Station #10	7.9	Strike Slip	2.74	329
RSN4040	Bam, Iran	2003	Bam	6.6	Strike Slip	1.70	487

5. Evaluation

Plastic hinge formation in the original frames and PBDP frames are extracted from nonlinear dynamic analysis and presented in the following figures. As can be seen, in the PBDP frames plastic hinges in beams are extended from the lower story to almost top level that meant the excellent contribution of all yield members to dissipate the earthquake input energy. In contrast, the contributions of beam rotation along the height of the original frames are limit and elastic deformation are concentered in the middle stories as compared with the PBPD frames.

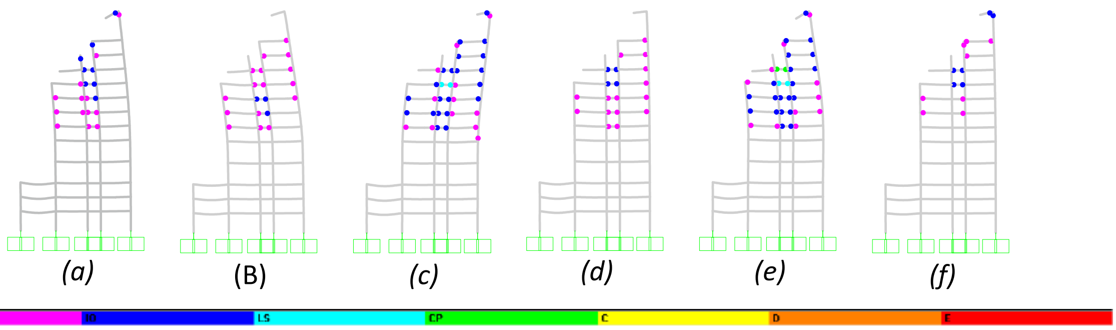


Figure1-Plastic hinge development in the low rise irregular frames, (a),(c),(e) plastic design methodology (b),(d),(f) and elastic design method.

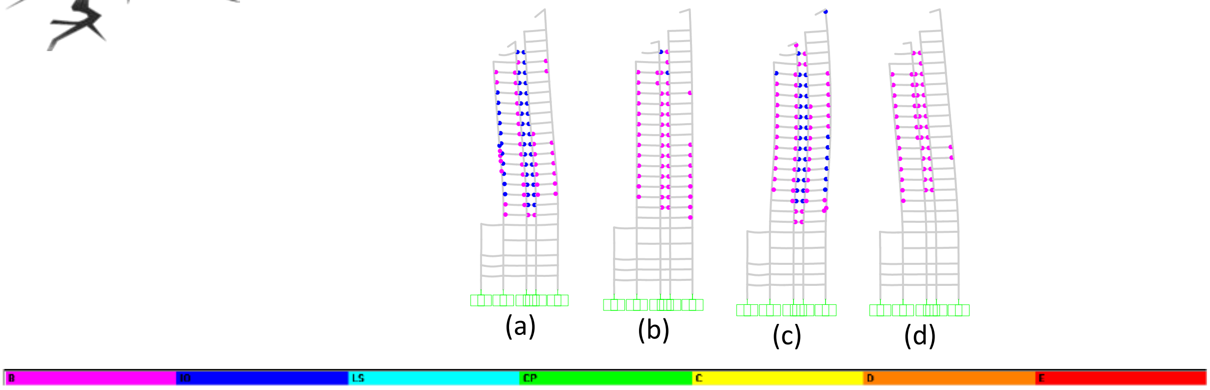


Figure2-Plastic hinge development in the mid rise irregular frames, (a),(c) plastic design methodology (b), (d) elastic design method.

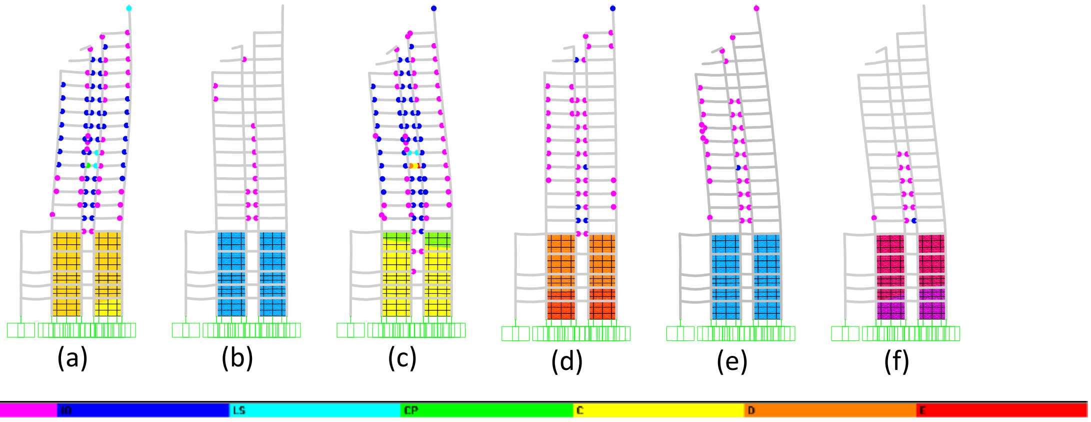


Figure3-Plastic hinge development in the mid rise irregular dual systems, (a),(c),(e) plastic design methodology (b), (d),(f) and elastic design method.

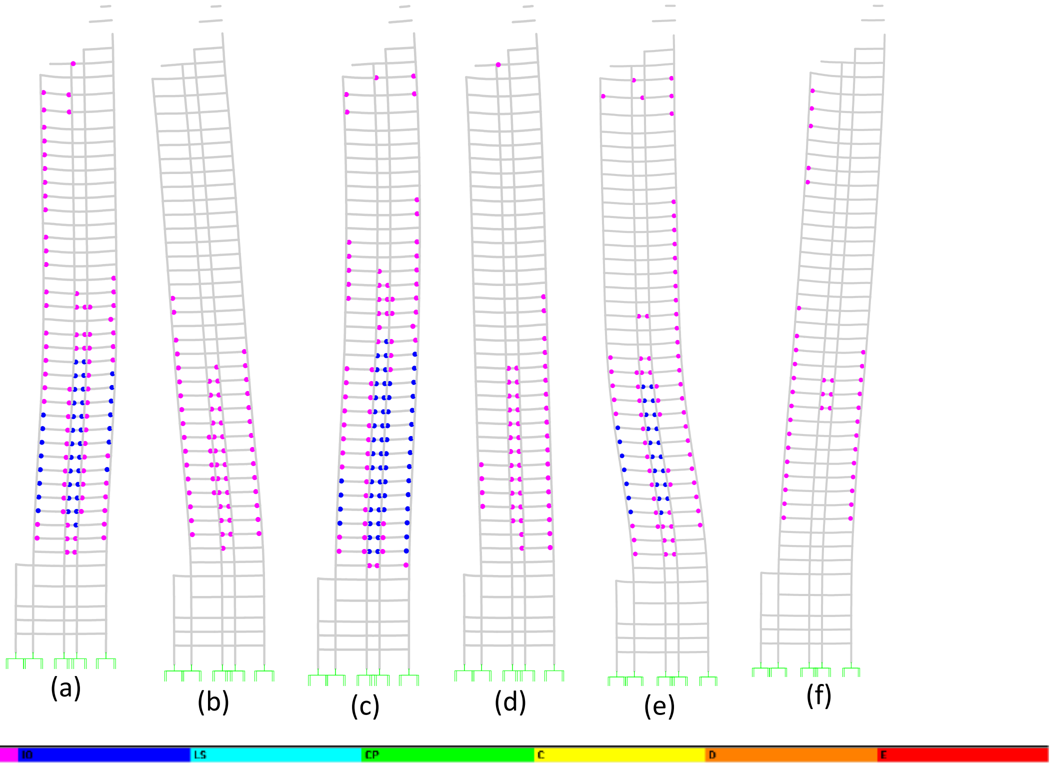


Figure4-Plastic hinge development in the low rise irregular frames, (a),(c),(e) plastic design methodology (b), (d),(f) and elastic design method.

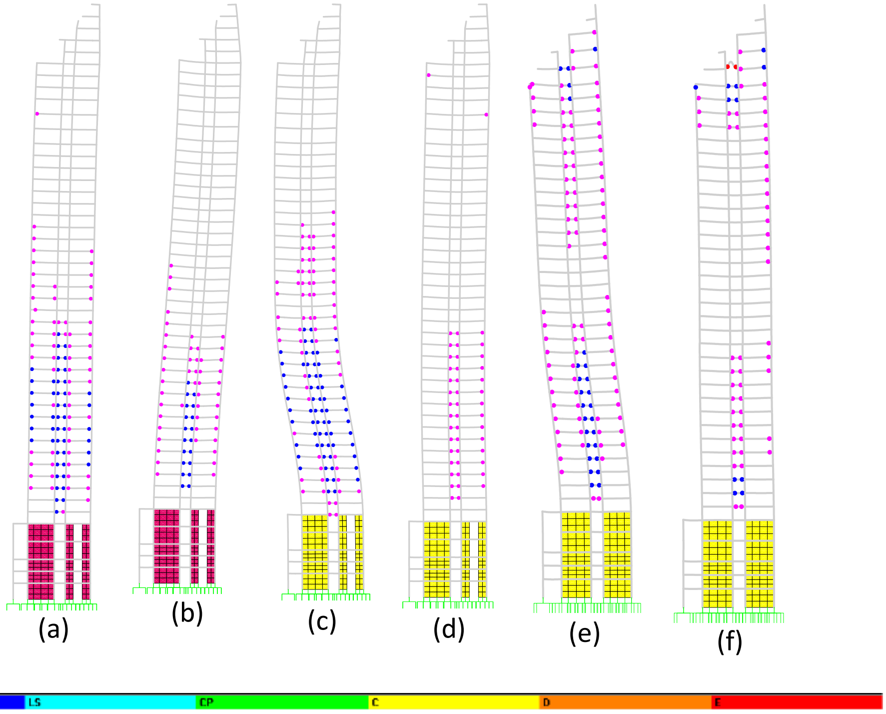


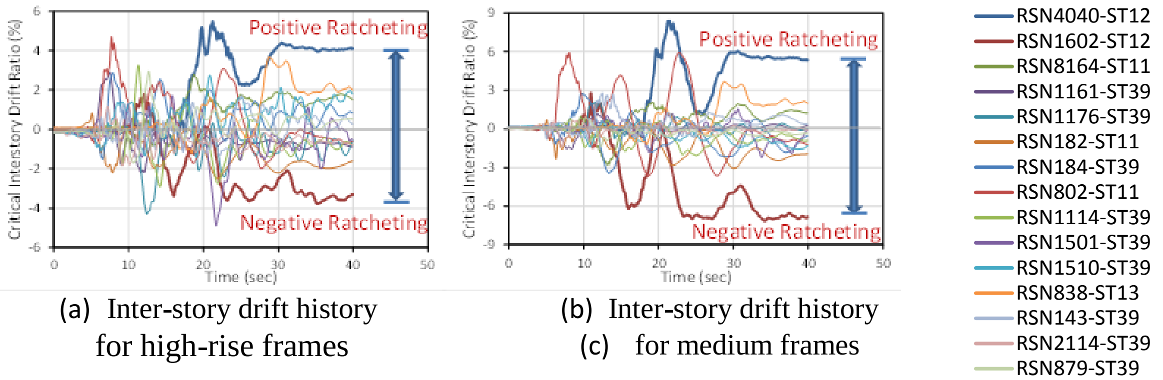
Figure5-Plastic hinge development in the low rise irregular frames, (a),(c),(e) plastic design methodology (b), (d),(f) and elastic design method.

6. Collapse assessment of the frames

The methodology which was developed by FEMA P695 is adopted herein for collapse evaluation of the high rise irregular frames designed by the PBPD. In this methodology two levels of ground motion are considered as; (1) collapse level ground motion which causes median collapse (2) MCE ground motion demand level.

For the derivation of fragility curve the IDR is used as the engineering demand parameter (EDP). First mode spectral acceleration is also chosen as the seismic intensity parameter. Ground motion intensities are increased for each record until dynamic instability occurs as a result of sudden increase in EDP. TBI declares that the mean of the absolute values of the maximum transient drift ratios from the set of analyses in each story level shall be less than 3%. Additionally, the absolute value of the maximum story drift ratio from the set of analyses shall not exceed 4.5%.

In figure 6, critical inter-story drift histories due to set of ground motions scaled to collapse intensities are illustrated. As can be seen in figure 6, almost all records meet the TBI criteria with maximum drift ratios level 3%.



(a) Inter-story drift history for high-rise frames

(b) Inter-story drift history (c) for medium frames

Figure 6- Critical inter-story drift history for different ground motions scaled to incipient collapse intensities

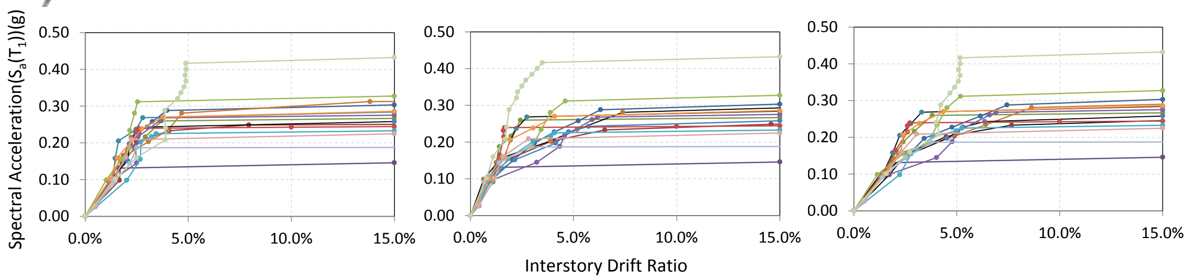


Figure 7- Incremental dynamic analysis results (a) IDA curves for 50 stories irregular moment frames (b) IDA curves for high rise irregular dual system (c) IDA curves for 30 stories irregular moment frames

As can be seen in Figure 7, structural performance for each ground motion favorably indicates safe margin against collapse as the maximum IDRs obtained from each records do not exceed 4.5%.

7. Summary and conclusions

Various steel moment-frames with the setback irregularity which assumed located at high earthquake-prone region are designed using current elastic method in accordance with ASCE7-10 and their results were compared with those obtained from the energy based plastic method meeting the requirements of the energy concept based method.

The main aim of study was to assess the capability of the energy concept based method for proportioning and detailing of irregular MRF system to withstand against collapse since irregular buildings currently lack a codified procedure to ensure the favorable mechanism. To investigate which approaches are more capable to dissipate the input earthquake energy through the formation of plastic hinges at their yielding members, nonlinear dynamic analyses were carryout for the analytical model of the original frames as well as PBDP frames and was subjected to a suite of spectral-matched earthquake ground motions with intensities from MCE hazard level to incipient collapse level. Monotonic and hysteretic modeling behaviors for different components including steel beams, columns and panel zones were obtained by adopting recommendations in TBI guidelines and PEER/ATC72 specifications.

As predicted, the desirable mechanism was developed in the PBPD frames by distribution of plastic hinge with sufficient rotation capacity on almost all beams. Despite the original frame were proportioned to meet the requirements of the current building code, distribution of plastic hinges in a few members represent that just a few members reached a fully plastic deformation while more than half remain elastic.

Thus, the effectiveness of the most yield member’s in dissipating earthquake energy as well as concentration of inelastic deformation in a few members which might reflect lack of rotational capacity in specific members as well as discontinuous load path distribution after yielding. In addition, seismic response of original frames indicates that whole capacity of the frames did not mobilize to contribute to the inelastic deformation meant the using current code-specified requirements for high rise irregular frames, May not lead to economic structures.

As part of the study, some controlling criteria reported in TBI in terms of maximum/mean IDRs, RDRs, and plastic rotations for MCE hazard level are compared with those obtained from NLRHA. Consequently, Structural acceptance criteria based on the requirements of TBI Guidelines, for MCE hazard level, in terms of maximum/mean IDRs, RDRs and plastic rotations as local parameters are reasonably satisfied.

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