RATIONAL USE OF INELASTIC RESPONSE IN SEISMIC DESIGN

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ABSTRACT :

Inelastic analysis procedures effectively accounts for several sources of force reduction. They are therefore more dependable means for predicting inelastic demands compared with elastic analysis procedures. Encouragement to employ the former procedures is still limited and designers tend to favor elastic procedures. The significant reduction allowed in response parameters obtained from elastic analysis procedures unlike those from inelastic analysis results in high uncertainties and discourages the effective exploitation of the latter procedure in design. The present study proposes a simple and theoretically-based approach that utilizes inelastic seismic response to refine the initial structural design. To effectively describe the proposed design approach, correlation of seismic demands obtained from different analysis procedures carried out using a comprehensive set of reinforced concrete buildings of different characteristics is investigated. Verified analysis tools and rational input ground motions are employed in the elastic and inelastic simulations. The benefits obtained from assessing the preliminary design using pushover analysis to determine the need for additional inelastic simulations are discussed. The presented approach enables engineers to arrive at a realistic and cost-effective design without compromising safety.

KEYWORDS: Seismic design, inelastic response, ductility, overstrength, RC buildings.

1. INTRODUCTION

Three-dimensional Elastic Analysis Procedures (EAPs) have become a relatively simple and rapid means of comparing initial estimates of seismic demands during the design stage of multi-story buildings. Modern seismic codes (e.g. ASCE 7, 2005; EC8, 2004) permit scaling down the strength demands obtained from such procedures by exploiting ductility and reserve strength to achieve an economical design (Mwafy and Elnashai, 2002; Elnashai and Mwafy, 2002). The deformations estimated from reduce seismic forces are then amplified to arrive at an estimate of the maximum inelastic deformation. This approach of estimating design actions and deformations is justified by the satisfactory performance of buildings designed to seismic codes during earthquakes, especially with regard to life safety. Inelastic Analysis Procedures (IAPs), in contrast, effectively account for several sources of force reduction, such as the redistribution of forces in the post-elastic range. Hence, they are a more dependable means for predicting inelastic demands for the design process. Moreover, code-recommended force reduction and displacement amplification factors used in EAPs are based on engineering judgment and have at best weak provenance (FEMA 450, 2003). Notwithstanding, encouragement to employ IAPs is limited, and designers tend to favor EAPs since significant reductions are allowed in response parameters obtained from elastic analysis unlike those from IAPs (e.g. Mwafy et al., 2006; FEMA 451, 2006).

The direct use of IAPs to refine and optimize the initial seismic design was investigated in a number of studies (e.g. Kappos and Manafpour, 2001; Vasilopoulos, and Beskos, 2006). The common idea in these approaches is the use of performance criteria corresponding to well-defined performance levels (e.g. immediate occupancy and life safety). Inelastic analysis is performed using input ground motions scaled to intensities corresponding to the selected performance levels. It is noteworthy that the explicit optimization approaches, which involve several iterations to select member cross-sections and reinforcement, are not addressed in the current study. These approaches are computationally demanding and may not be suitable for the design office environment. It is also important to note that the sole-use of IAPs in design is impractical since inelastic modeling requires detailed information about structural members, which cannot be obtained without an initial design using EAPs. The objectives of the present study is thus twofold:

- Discuss the uncertainties arising from the significant reduction allowed in response parameters obtained from EAPs unlike those from IAPs, which discourage the exploitation of the latter procedures in design.
- Propose a simple and theoretically-based design approach that utilizes the design overstrength and inelastic response for refining the initial design.

A comprehensive set of medium and high-rise buildings of different characteristics are reliably modeled for elastic and inelastic analysis. Rational input ground motions are selected based on site-specific characteristics and recommended design criteria. Different elastic and inelastic analysis procedures recommended by modern seismic codes are carried out using verified analysis tools to correlate their seismic demand. The benefits obtained from assessing the preliminary design using a simple response measure to determine the need for additional refined analysis are discussed and a rational design approach is finally presented.

2. ANALYSIS PROCEDURES FOR SEISMIC DESIGN

The Equivalent Lateral Force Procedure (ELFP) mainly accounts for response in the fundamental mode of vibration, which has lower modal mass compared with the total mass of the structure. Seismic codes attempt to conservatively account for higher modes through the seismic coefficient, Cs, and the vertical distribution of equivalent seismic forces. However, the use of Elastic Dynamic Analysis Procedures (EDAPs) are generally recommended for irregular and long period structures since they will not only result in a more realistic characterization of the distribution of inertial forces, but may also result in reduced seismic force demands (FEMA 450, 2003). EDAPs are classified as either Response Spectrum (RSP) or Linear Response History Procedure (LRHP). As explained in Figure 1, modern seismic codes permit scaling the response parameters obtained from EDAPs using the response modification factor (R). A lower bound is typically imposed by comparison with demands obtained from ELFP. If the building responds elastically to the design earthquake, the calculated internal forces from EAPs will be reasonable estimates of those expected during the design earthquake. It is also implicitly assumed that if the building responds inelastically to the design earthquake, as will commonly be the case, the internal forces that would develop in the building will be less than those calculated elastically. EDAPs are therefore allowed for all classes of structure regardless of the period or the degree of irregularity (e.g. ASCE 7, 2005; FEMA 450, 2003).

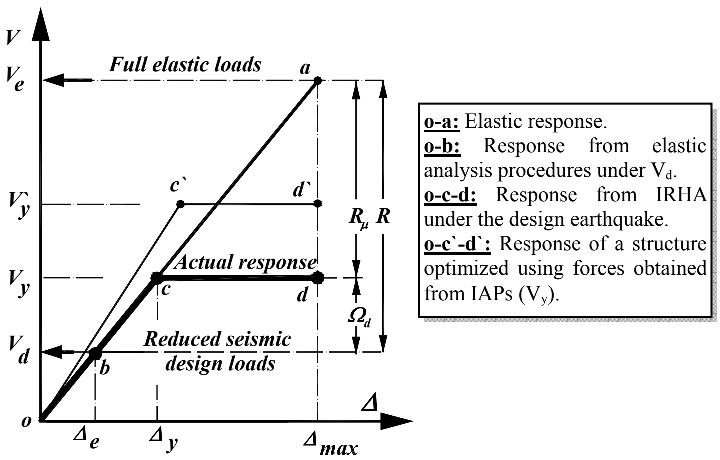

Figure 1. Different scenarios of lateral response.

Although the Nonlinear Response History Procedure (NRHP) is also permitted for all structural configurations, little encouragement is given to designers to use these procedures even for long period or irregular structures. Moreover, seismic codes do not permit any reduction of response parameters obtained from NRHP. This approach is justifiable if NRHP is used to verify the design acceptability and check the actual demand imposed on the structure versus capacity. However, refining the initial design is frequently required when designing an important structure or when a more cost-effective design is desired. The code-approach of using actual strength (V_y) obtained from IAPs to refine the initial design is over-conservative since overstrength is inevitably added during the design process (Elnashai and Mwafy, 2002). Since the design overstrength (Ω_d) is defined as the ratio of the actual to design lateral strength ($\Omega_d = V_y / V_d$), the strength used in design can be estimated by scaling down the actual strength obtained from IAPs using the overstrength factor ($V_d = V_y / \Omega_d$). It is clear from Figure 1 that base shear demands from NRHA (V_y) is much higher than those obtained from EAPs (V_d) due to the existence of Ω_d. Employing V_y to optimize the initial design of the structure results in higher reinforcement and larger cross sections (line o-c`-d` in Figure 1) compared with the initial design (line o-c-d). This issue causes high uncertainties in design since the response parameters obtained from EAPs are permitted to be reduced using the R factor unlike those from IAPs. These observations are confirmed from the results presented hereafter.

3. STRUCTURAL SYSTEMS, EARTHQUAKE LOADING AND ANALYTICAL MODELING

Sixteen medium and high-rise buildings with a wide range of configurations and structural systems were selected for the current study. The characteristics of these Reinforced Concrete (RC) buildings are given in Table 1. It is clear that the selected structures can be classified, according to their period of vibration and total number of stories, into two main groups. The eight structures in group A were selected from the modern building stock in Dubai to represent characteristics of contemporary high-rise RC buildings designed to modern seismic codes. Different building heights (29-54 stories) and structural systems (shear walls and dual systems) were selected. The lateral force design was conducted according to UBC (1997), while sizing and detailing of structural members were undertaken based on the ACI regulations (ACI, 2002). Despite the fact that ASCE 7 (2005) are the official seismic design codes for buildings in the US, UBC (1997) is still extensively employed in several regions outside the US. The lateral force resisting systems of this set comprise of RC cores, shear walls, columns and floor slabs. The foundation system comprises of a system of piles supporting a rigid RC raft. Normal-to-high strength materials were used in design (40-60 N/mm^2 for concrete and 460 N/mm^2 for steel).

The eight buildings in group B were selected to represent contemporary medium-rise RC buildings (8-12 stories) designed to Eurocode 8 (Fardis, 1994). The selection of this group of buildings was motivated by the desire to include in the study a sample of structures carefully designed and detailed to the modern design practice outside the US. The eight buildings in Group B may be subdivided into two subsets based on their height. The four buildings in each subset were designed to three design ductility levels (Low, Medium and High) and two design ground accelerations (0.15g and 0.30g), which lead to four cases within each subset. Characteristic strength for concrete and yield strength for steel of 25 N/mm^2 and 500 N/ mm^2, respectively, were used in the design.

The investigated buildings are assessed under the effect of seismic scenarios representing the design earthquake The seismic risk at this level should satisfy the requirements of the life safety performance level. Based on conclusions of a comprehensive hazard study undertaken for Dubai (Mwafy et al., 2006), five synthetically generated accelerograms and two natural records (Emeryville, USA, 1989 and Hollister City Hall, USA, 1974) were selected for analysis of Group A buildings. This follows recommendations of seismic codes (ASCE 7, 2005; UBC, 1997) to allow employing average response from the seven records. The selected records represent two distinct seismic scenarios: (i) the first scenario represents severe earthquakes of a magnitude 7.4 with 100 km epicentral distance and (ii) the second is for moderate events of a magnitude 6.0 and a shortest distance to causative fault of 10 km. The records were scaled to a design PGA of 0.16g recommended for Dubai for a 10% probability of exceedance in 50 years. It is noteworthy that scaling site-specific input ground motions according to a smoothed design spectrum, as recommended by seismic design provisions, is illogical since it eliminate the most important characteristics of ground motions (FEMA 451, 2006). Furthermore, analyses of Group B buildings are performed using six input excitations; four artificially-generated records compatible with the design code response spectrum and two natural earthquake records (Kobe 'KBU', Japan, 1995 and Loma Prieta 'SAR', U.S., 1989). The selected two natural earthquake records are also applied with and without the vertical component of ground motion, which lead to a total of eight different input excitations used in analysis. Since Group B buildings were designed according to EC8, the selected records were scaled to possess equal velocity spectrum intensity in the period range of the buildings. Further information regarding the scaling approach and analytical modeling can be found elsewhere (Mwafy and Elnashai, 2001 & 2008). Comparisons of response spectra of input ground motions used in analysis of Group A and Group B buildings are shown in Figure 2.

Table 1. Characteristics of the investigated buildings and their period of vibrations.

Group	Building	No. of stories	Structural system-design ductility-design PGA (g)	Longitudinal dir.		Transverse dir.	
				T_1(sec.)	T_2(sec.)	T_1(sec.)	T_2(sec.)
A	TR1	54	FW – Medium - 0.16	3.7	1.2	4.1	1.2
	TR2	53	FW – Medium - 0.16	3.9	1.1	4.6	1.2
	TR3	53	FW – Medium - 0.16	3.8	1.1	4.9	1.3
	TR4	50	FW – Medium - 0.16	2.9	0.9	3.9	1.0
	TR5	45	SW – Medium - 0.16	3.1	0.9	3.5	0.9
	TR6	44	FW – Medium - 0.16	3.4	0.9	3.9	1.1
	TR7	39	SW – Medium - 0.16	3.6	0.9	3.6	1.0
	TR8	29	FW – Medium - 0.16	1.8	0.5	2.1	1.5
B	TR9	12	MRF – High - 0.30	0.73	0.24	0.83	0.27
	TR10	12	MRF – Medium - 0.30	0.76	0.25	0.87	0.28
	TR11	12	MRF – Medium - 0.15	0.78	0.25	0.89	0.29
	TR12	12	MRF – Low - 0.15	0.78	0.25	0.89	0.29
	TR13	8	MRF – High - 0.30	0.58	0.19	0.63	0.20
	TR14	8	MRF – Medium - 0.30	0.58	0.19	0.63	0.20
	TR15	8	MRF – Medium - 0.15	0.66	0.22	0.72	0.23
	TR16	8	MRF – Low - 0.15	0.66	0.22	0.72	0.23

FW: Dual frame-wall systems. *SW: Shear wall systems.* *MRF: Moment-resisting frames.*

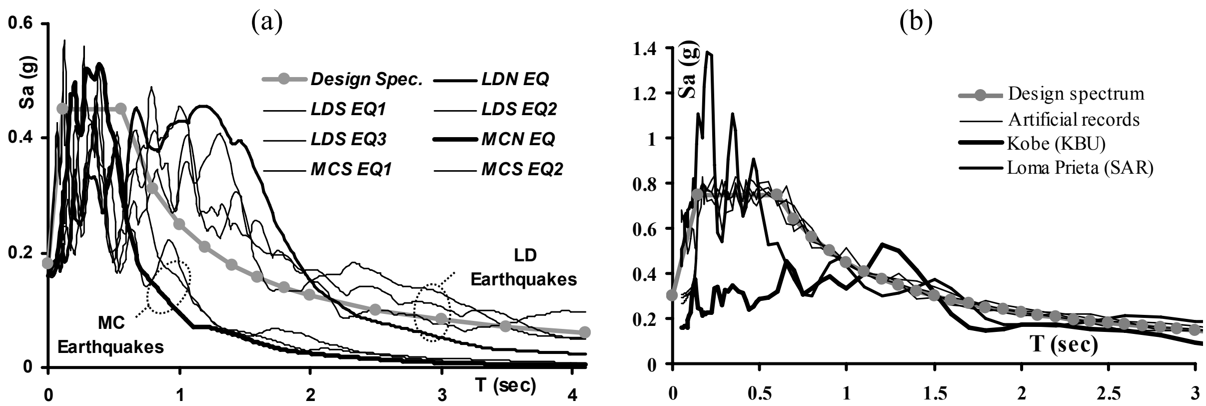

Figure 2. (a) Response spectra (5% critical damping, PGA = 0.16g) of UBC-97, Large Distant Natural (LDN), Large Distant Synthetic (LDS), Moderate Close Natural (MCN) and Moderate Close Synthetic (MCS) ground motions used in analysis of Group A buildings; (b) Response spectra (5% critical damping, PGA = 0.3g) of EC8, artificial and natural ground motions used in analysis of Group B buildings.

The structural analysis program used for the three-dimensional elastic analysis is the program ETABS (CSI, 2003). The buildings were modeled with cracked stiffness and equivalent viscous damping of 5%, which is the damping value expected for loading near the yield point. On the other hand, refined idealizations of the buildings were adopted for inelastic analysis using the program Zeus-NL (Elnashai et al., 2006). Each structural member is assembled using a number of cubic elasto-plastic elements capable of representing the spread of inelasticity within the member cross-section and along the member length via the fiber analysis approach. The stress-strain response at each fiber is monitored during the entire multi-step analysis. The expected (mean) material strengths are used in the inelastic analysis. Although Zeus-NL is effectively capable of performing three-dimensional NRHP of multi-story structures using the fiber modeling approach, such analysis are computationally demanding, particularly for high-rise buildings. Therefore, a two-dimensional idealization is utilized for inelastic analysis of the buildings. Moreover, to reduce the extensive modeling effort and amount of the inelastic analyses, TR1 was selected as a representative of Group A buildings. All buildings in Group B were modeled and analyzed using Zeus-NL. The sample inelastic analysis results presented in subsequent sections are for the transverse direction of TR1 and the longitudinal direction of TR9-TR16, which are the critical directions of the buildings. Figure 3 depict the three-dimensional analytical models of the buildings investigated in this study, which were used to predict the response of all buildings shown in Table 1 using IAPs.

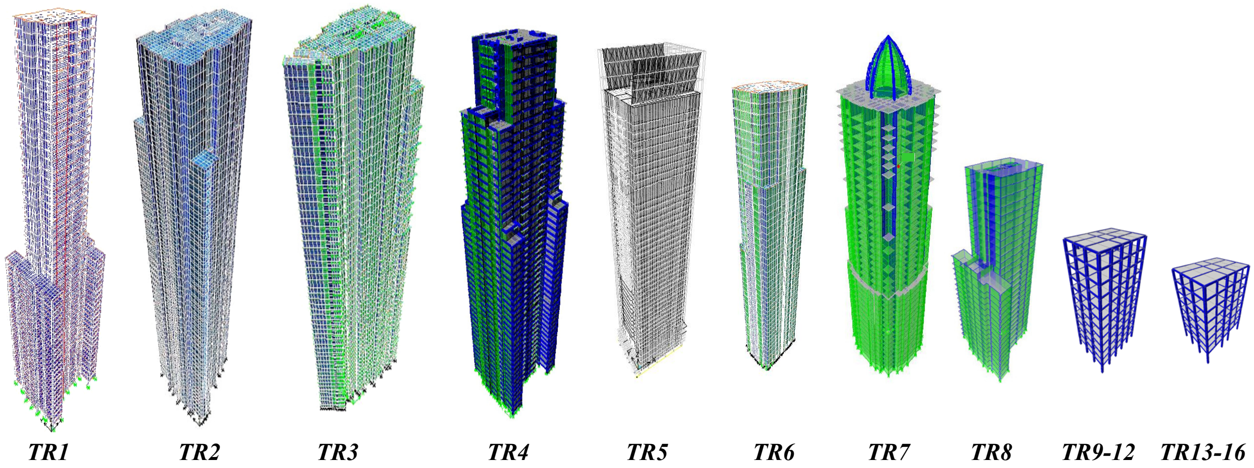

Figure 3. Three-dimensional analytical models of buildings.

4. COMPARISON OF ELASTIC AND INELASTIC ANALYSIS RESULTS

4.1. Elastic Analysis Procedures

Sample results from these analyses are shown in Table 2 for Group A buildings. It is clear that ELFP is more conservative and produces higher design base shear compared with RSP. The over-conservatism of ELFP increases with increasing height (from TR8 to TR1). It is clear that the lower limit imposed by the design code (UBC, 1997) for the reduction of response parameters obtained from RSP (90% of those from ELFP) should be utilized in design of all buildings investigated. Notwithstanding, the results confirm that ELFP may be non-conservative in predicting base shear demands when employing the site specific spectra in RSP, particularly if the site is dominated by a single seismic scenario from large distant events. It is clear from Table 2 and Figure 2(a) that response results of TR1 and TR8 obtained from RSP using Emeryville are higher than ELFP due to the high amplification in the period range 1.0 to 1.8 sec, which corresponds to the second mode period of these buildings.

Table 2. Comparison of total base shear (V, kN) of Group A buildings from ELFP and RSP.

Building	Equivalent lateral force analysis	Response spectrum analysis [a]									
		Design Spectrum		Max. demand [a]		Min. demand [a]		Mean [a]		Median [a]	
		Long. dir.	Trans. dir.	Long. dir.	Trans. dir.	Long. dir.	Trans. dir.	Long. dir.	Trans. dir.	Long. dir.	Trans. dir.
TR1	18368	11180	12454	15740	**20555**	4021	4658	7378	9075	5080	6636
TR2	21761	11611	11013	13210	15524	4119	4168	7081	7445	5572	5556
TR3	21653	11356	10552	11866	14622	3932	4256	6717	7454	5139	5925
TR4	22153	16201	14661	20516	19574	5943	6453	10323	10886	7605	8163
TR5	19123	13327	12758	15867	15720	6776	7688	9384	10134	7840	8567
TR6	12543	8385	7757	10905	9669	3834	3256	5753	5355	4397	3921
TR7	13259	9307	8306	11170	11170	4560	3119	6701	5738	5559	4376
TR8	13582	12288	10326	**16268**	10258	5001	3746	8997	6872	7273	5663

a: Demands from seven site specific input ground motions employed in response spectrum analysis.

4.2. Inelastic Analysis Procedure

A summary of seismic demands from the extensive inelastic analyses of TR1 and TR9-16 is shown in Table 3. Comparison of NRHP results with the design base shear calculated from elastic analysis procedures confirms that results of NRHP are significantly higher than EAPs for all buildings. The base shear of TR1 from the conservative ELFP is only 20% of that from NRHP. The same observation applies, with less extent, to the Group B buildings. On the other hand, the code approach concerning the deformation demands obtained from NRHP is consistent and justifiable from the design point of view. For instance, the maximum interstory demand of TR1 from ELFP is 0.22%. According to the design code (UBC, 1997) this drift demand should be amplified by a factor of 0.7R to arrive at the maximum inelastic displacement (0.85%). This demand is comparable to that obtained from inelastic analysis (0.9%). Clearly, the code approach results in consistent deformation demands from different analysis procedure unlike force-related parameters.

Table 3. Assessment of response for refining the initial design.

Ref.	Base shear (kN)		Ω_i	R	Classification (refer to Figure 4)	Recommendation	Design base shear from NRHP scaled using R_{in}
	NRHP[a]	NRHP[a] / Design					
TR1	88749	4.83	1.0	5.5	Uneconomical Design	Redesign	20170
TR9	11105	2.37	0.48	5	Middle Zone	Acceptable	-
TR10	11316	1.89	0.55	3.75	Middle Zone	Acceptable	-
TR11	5116	1.78	0.64	3.75	Middle Zone	Acceptable	-
TR12	5661	1.3	0.85	2.5	High Overstrength Zone	Possible saving	3330
TR13	9007	2.24	0.59	4	Middle Zone	Acceptable	-
TR14	9168	1.71	0.73	3	Middle Zone	Acceptable	-
TR15	4908	1.93	0.84	3	High Overstrength Zone	Possible saving	2435
TR16	4722	1.24	1.05	2	Uneconomical Design	Redesign	2811

a: Mean demands are from a suite of at least seven input ground motions.

The results confirm the conclusions drawn above regarding the over-conservative base shear demands form NRHA (V_y) for design. This is mainly due to the existence of design overstrength (Ω_d), as explained in Figure 1. The rational approach is therefore to reduce V_y using the actual overstrength (Ω_d) of the structure to arrive at a consistent and cost-effective design base shear from different analysis procedures. This issue has also been recently raised by FEMA 451 (2006). Elastic and inelastic analysis procedures were carried out to estimate the design forces of a six-story steel frame building. The results indicated that the base shear from NRHP was more than four times the values calculated from ELFP. The results presented in the current study and elsewhere confirm the pressing need for a reliable approach to support the use of inelastic analysis procedures in design and to clarify the apparent problem, whereby inelastic response history analysis is grossly over-conservative.

5. REFINEMENT OF INITIAL SEISMIC DESIGN USING INELASTIC ANALYSIS

A simple measure of response termed 'inherent overstrength factor' (Ω_i), which compares the ultimate strength of the structure (V_y) with the elastic design force (V_e), was suggested by Elnashai and Mwafy (2002). To estimate the inherent overstrength factor, pushover analysis is conducted using Zeus-NL. Although several improvements have been recently suggested to advance pushover analysis, newly developed procedures still do not guarantee satisfactory results with increasing the structural irregularity and input ground motion peculiarity. The enhancements involved in new proposals have also some effects on simplicity (Maison, 2005), which is an essential requirement for the analysis procedures intended for design. Conventional pushover analysis has been employed for capacity estimates of tall buildings and highway bridges (e.g. Mwafy et al., 2006; Mwafy et al., 2007). It is therefore suggested by Mwafy et al. (2006) and Mwafy and Elnashai (2001) to employ a uniform and inverted triangular lateral load distributions to conservatively estimate the ultimate capacity of high-rise and medium-rise buildings, respectively. Table 3 shows the average inherent overstrength factor (Ω_i) of the investigated buildings, while Figure 4 shows a pictorial view of the buildings response measured using Ω_i. The results indicate that the predicted seismic response of TR1 and TR16 using the inherent overstrength factor is elastic under the design earthquake. This conclusion is also confirmed from inelastic member response of the investigated buildings.

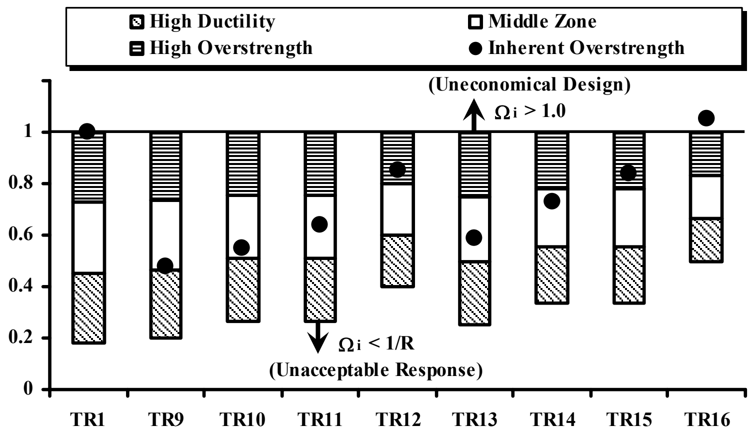

Figure 4. Assessment of response using inherent overstrength.

Figure 4 also shows clearly the higher overstrength of the 8-story irregular buildings (TR13-TR16) compared to the 12-story regular structures (TR9-TR12). EC8 adopts more conservative R factors for irregular structures, which results in higher overstrength. Moreover, for the buildings designed to the same PGA (e.g. TR11-TR12), the higher ductility level buildings (e.g. TR11) show lower Ω_i, reflecting the higher response in the post-elastic range. Finally, the values of Ω_i are realistic in terms of the higher overstrength of buildings designed to lower PGA since gravity loads are more influential compared with lateral loads. Clearly, the inherent overstrength is a rational tool and enables characterizing the response of the structure with reference to a well-defined limit state.

Based on the abovementioned results, it is suggested to adopt the inherent overstrength factor for refinement of the initial design. Figure 5 summarizes the proposed approach. Pushover analysis is only needed to estimate Ω_i, which effectively reflects the anticipated behavior of the structure under the design earthquake. NRHA is recommended if Ω_i indicates unacceptable or uneconomical response. To rationally exploit results of inelastic analysis procedures in design, a realistic reduction factor of base shear demands is proposed. This scaling factor is mainly the actual overstrength. A safety factor (β) is recommended to account for uncertainties associated with estimating the inherent overstrength factor using pushover analysis and to insure the conservatism of the suggested approach. Sensitivity analyses are required to calibrate this safety factor. For the sample of buildings investigated in the present study, the suggested safety factor is 0.8. Additional studies are needed to calibrate this safety factor for different structural systems and configurations. It is worth noting that seismic codes (e.g. ASCE 7, 2005; UBC, 1997) also impose empirical limitations on the reduction of base shear obtained from dynamic analysis procedures. These limitations are justified by several uncertainties in modeling and analysis (FEMA 450, 2003). The above discussion results in a reduction factor for inelastic analysis procedures $R_{in} = \beta \,.\, \Omega_d = \beta \,.\, R \,.\, \Omega_i$.

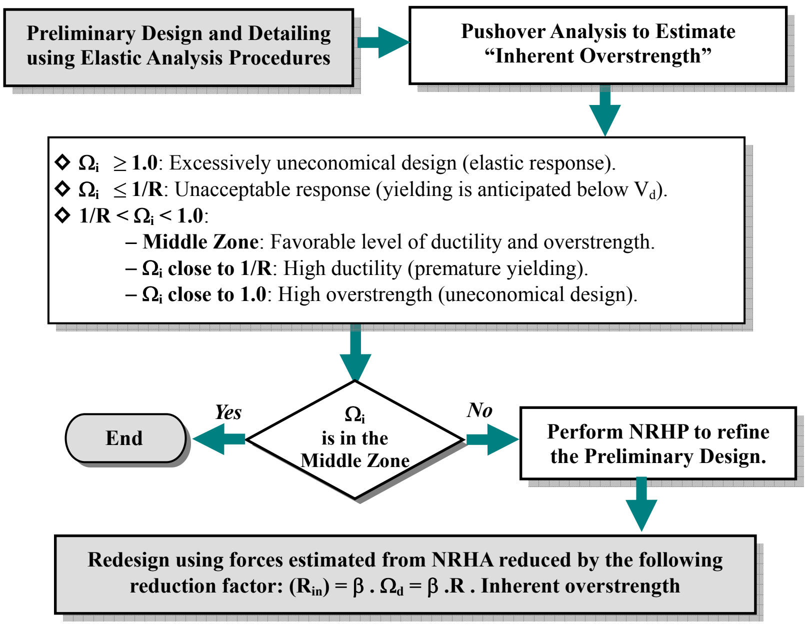

Figure 5. Refine the initial design using IAPs.

Table 3 shows application of the suggested approach for refining the design of tower TR1 and TR9-16. The theoretically-based approach presented in the present study exploits the actual ductility (accounted for in NRHA) and design overstrength (Ω_d) to refine the design without jeopardizing safety. It is believed that NRHA will play a more influential role in seismic design of structures as a result of the rapid advances in inelastic analysis platforms. It is also noteworthy that an extensive effort is currently underway to refine and update modern seismic provisions (e.g. ATC-63, 2008). This effort is intended to recommend response parameters for use in seismic design and improve safety against collapse. The effort presented in the present study is intended to support ongoing activities towards having different structures with similar reliability, an objective not fully achieved by existing code design procedures.

6. CONCLUSIONS

Different elastic and inelastic analysis procedures recommended by modern seismic codes were undertaken for a wide range of medium- and high-rise buildings to compare their results, considering all feasible seismic scenarios. Advantages and drawbacks of each procedure for analysis of this diverse range of buildings were discussed. Correlation of demands from elastic and inelastic procedures confirmed the over-conservatism of the latter procedure for refining the initial design as currently suggested by seismic codes. Inelastic deformation demands were comparable from different analyses, while base shear demands from Equivalent Lateral Force Procedure (ELFP) and Response Spectrum Procedure (RSP) were significantly lower than those from Nonlinear Response History Procedure (NRHP). Use of the latter procedure is therefore hampered by the over-conservative requirements that inhibit any reductions in inelastic demand estimates for design. It is suggested to use a simple measure of response for refining the initial design. Conventional pushover analysis is only needed to estimate this measure, which directly reflects the anticipated behavior of the structure under the design earthquake. NRHA is recommended to refine the design if the measure indicates a response outside the favorable range. A reduction factor of base shear demands is proposed to account for the design overstrength and rationally exploit results of NRHP in design. Although the proposed approach is currently at the state of development, it offers a simple and theoretically-based approach to refine the initial design. The approach exploits the actual ductility (accounted for in NRHA) and overstrength to arrive at a realistic and cost-effective design without compromising safety.

ACKNOWLEDGMENTS

The author is indebted to Professor A.S. Elnashai, Director of Mid-America Earthquake Center, USA, for his valuable comments on this work. This help is gratefully acknowledged.

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