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Efficient and cost-effective seismic design using inelastic analysis procedures and overstrength

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ABSTRACT: This paper discusses the advantages of a simple approach that utilizes inelastic analysis procedures (IAPs) in seismic design. Two sets of medium- and high-rise buildings of different structural systems and heights are employed to describe and verify the approach. Fiber-based numerical models and diverse input ground motions are employed in the dynamic response simulations. The correlation of the results obtained from different IAPs and elastic analysis procedures (EAPs) suggests utilizing a simple measure of response for assessing the initial design. This measure reliably describes the anticipated behavior of the structure under the design earthquake. Nonlinear response history analysis (NRHA) is recommended to refine the initial design if the response was outside a favorable range. The proposed refinement in seismic design effectively exploits IAPs and first yield overstrength, and helps designers to obtain more reliable and economical design of building with different structural systems and heights.

1 INTRODUCTION

The increase in construction of multi-story buildings around the world attracts focused research aiming to arrive at reliable and cost-effective design approaches. Modern seismic design codes (e.g. CEN 2004; ASCE-7 2010) permit decreasing the design strength obtained from EAPs by utilizing ductility and overstrength to arrive at a cost-effective design (Mwafy and Elnashai 2002). This approach is justified by the satisfactory performance of well-designed structures in previous earthquakes. On the other hand, the use of IAPs in design is limited despite the fact that different sources of force reduction are effectively accounted for inelastic analysis. The reduction in response parameters obtained from EAPs unlike those from IAPs discourages the effective use of the latter analysis procedures to refine the initial design of a wide class of structures such as irregular and long-period buildings. Previous studies highlighted the over-conservatism of strength demands obtained from IAPs if used in design (e.g. FEMA 2006; Mwafy 2008). Possible refinements in seismic design were also proposed in the latter study by utilizing the inelastic structural response obtained from IAPs. The present study is an extension of the initial work proposed by Mwafy (2008), which assessed a limited number of high-rise buildings using IAPs particularly those designed using U.S. codes. Fourteen medium- and high-rise buildings of different characteristics, ranging from 8 to 60 stories, are systematically idealized and assessed in the current study using EAPs and IAPs, aiming at: (i) refining the initial approach proposed by Mwafy (2008); and (ii) providing a final verification of this theoretically-based design approach to enable utilizing it in the design process.

2 DESIGN APPROACH OF MODERN SEISMIC CODES

The reserve strength (Ω_d) and the ability of the structure to dissipate energy are exploited by modern seismic codes to reduce the elastic seismic force (V_e) to arrive at a cost-effective design force (V_s), as shown in Figure 1 (FEMA 2009). Figure 2 shows sample of the capacity curves of reference structures, as subsequently discussed. The Equivalent Lateral Force Analysis (ELFA) mainly accounts for response in the fundamental mode of vibration. Seismic codes attempt to account for higher modes by shifting the resultant force of the equivalent seismic forces upwards, thereby increasing overturning effects. The use of Elastic Dynamic Analysis Procedures (EDAPs) is recommended for long period structures since they will not only result in a more realistic characterization of the distribution of inertial forces in the structure but may also result in reduced seismic force demands (FEMA 2009). EDAPs include Modal Response Spectrum Analysis (MRSA) and Linear Response History Analysis (LRHA). Seismic codes permit scaling down the response parameters obtained from EDAPs using the response modification factor (R). A lower bound is typically imposed by comparison with demands obtained from ELFA. EAPs are allowed for the design of all classes of structures regardless of the period or the degree of irregularity (ASCE-7 2010).

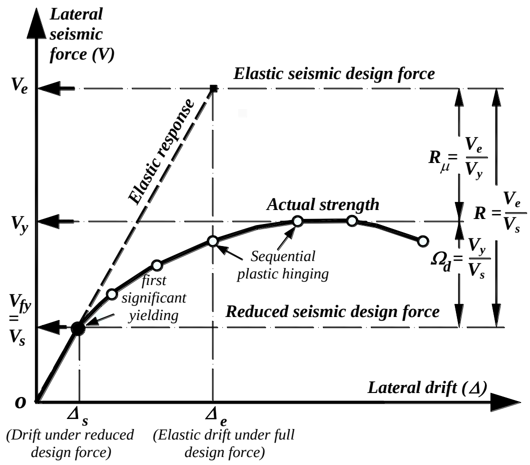

Figure 1. Lateral force-deformation response of a properly designed structure as suggested by design provisions.

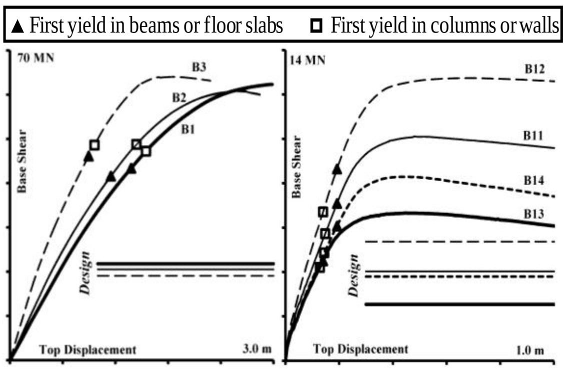

Figure 2. Tracing of the lateral force-deformation response of the reference structures (sample results).

The sole-use of NRHA in design might be impractical since inelastic modeling requires detailed information about structural members, which cannot be obtained without initial design using elastic analysis. NRHA are therefore utilized to refine the initial design of special structures such as irregular and long-period buildings. This analysis procedure might be compulsory in the simulation of highly irregular structures, critical buildings, and base isolation systems. Modern seismic codes do not permit the reduction of response parameters obtained from NRHA for design. Although this approach is justifiable if NRHA is used to verify the design acceptability and check the demand imposed on the structure against actual capacity, it is over-conservative in design since overstrength is inevitably added during the design process.

3 REFERENCE STRUCTURES AND NUMERICAL MODELLING

A wide range of medium and high-rise buildings with different heights and structural systems were selected for the present study, as shown in Table 1. The selected structures are classified into two main groups. The six structures in group A represent contemporary medium- to high-rise concrete wall buildings with different heights (10-60 stories). The lateral force design was conducted according to ASCE/SEI 7-10 (2010). The buildings were proportioned and detailed according to the ACI building code (ACI-318 2008). The eight buildings in group B were designed and detailed in accordance with Eurocode 2 and 8 to represent contemporary medium-rise frame buildings (8-12 stories). The selection of this group of buildings was motivated by the desire to include in the study a sample of structures designed to the design practice in Europe with various ductility and design Peak Ground Accelerations (PGA). The eight buildings in Group B are subdivided into two subsets based on their heights. The four buildings in each subset were designed to three design ductility classes (Low, Medium and High) and two PGA (0.15g and 0.30g). Figure 3 shows the layouts and 3D models of the buildings, while additional design information of the reference structures are presented elsewhere (Mwafy 2013)

Table 1. Characteristics of fourteen reference structures.

Group	Building reference	No. of stories	Total height (m)	Structural system	Design PGA (g)	Period in longitudinal dir.		Period in transverse dir.	
						T_1 (sec.)	T_2 (sec.)	T_1 (sec.)	T_2 (sec.)
A	B1	60	193.3	IBW	0.15	6.35	2.01	6.55	1.84
	B2	50	161.3	IBW	0.15	5.16	1.62	5.14	1.44
	B3	40	129.3	IBW	0.15	4.01	1.25	3.82	1.06
	B4	30	97.3	IBW	0.15	3.09	0.94	2.89	0.76
	B5	20	65.3	IBW	0.15	2.03	0.60	1.78	0.44
	B6	10	33.3	IBW	0.15	0.95	0.26	0.74	0.17
B	B7	12	36.0	SMRF	0.30	0.73	0.24	0.83	0.27
	B8	12	36.0	IMRF	0.30	0.76	0.25	0.87	0.28
	B9	12	36.0	IMRF	0.15	0.78	0.25	0.89	0.29
	B10	12	36.0	OMRF	0.15	0.78	0.25	0.89	0.29
	B11	8	25.5	SMRF	0.30	0.58	0.19	0.63	0.20
	B12	8	25.5	IMRF	0.30	0.58	0.19	0.63	0.20
	B13	8	25.5	IMRF	0.15	0.66	0.22	0.72	0.23
	B14	8	25.5	OMRF	0.15	0.66	0.22	0.72	0.23

IBW: Intermediate Bearing Wall system (medium level of ductility); SMRF: Special Moment-resisting frames (high level of ductility); IMRF: Intermediate Moment-resisting frames (medium level of ductility); OMRF: Ordinary Moment-resisting frames (low level of ductility)

Detailed idealizations are adopted for the inelastic analysis of the reference structures using Zeus-NL (Elnashai et al. 2012). Each structural member is assembled using a number of elasto-plastic frame elements capable of representing the spread of inelasticity within the member cross-section and along the member length via the fiber modeling approach. The concrete response is represented using a uniaxial constant confinement concrete model, while an elasto-plastic model is selected to represent the reinforcing steel (Elnashai et al. 2012). The mean (expected) material strength values are used in the inelastic analysis. Although Zeus-NL is capable of performing 3-D inelastic analysis of multi-story structures, such analysis is computationally demanding, particularly for high-rise buildings. Therefore, a two-dimensional idealization is utilized for inelastic analysis. The inelastic behavior of the Group A buildings indicated that the response is comparable in the longitudinal and transverse directions. This suggests that the number of inelastic analyses can be reduced by focusing on the transverse direction (i.e. framing system A). Moreover, the critical response of the long span beams of the Group B buildings supports undertaking the inelastic analysis in the longitudinal direction (i.e. framing system E and F). For the sake of brevity, additional information concerning the modeling of the reference structures for inelastic analysis is presented elsewhere (Mwafy 2013).

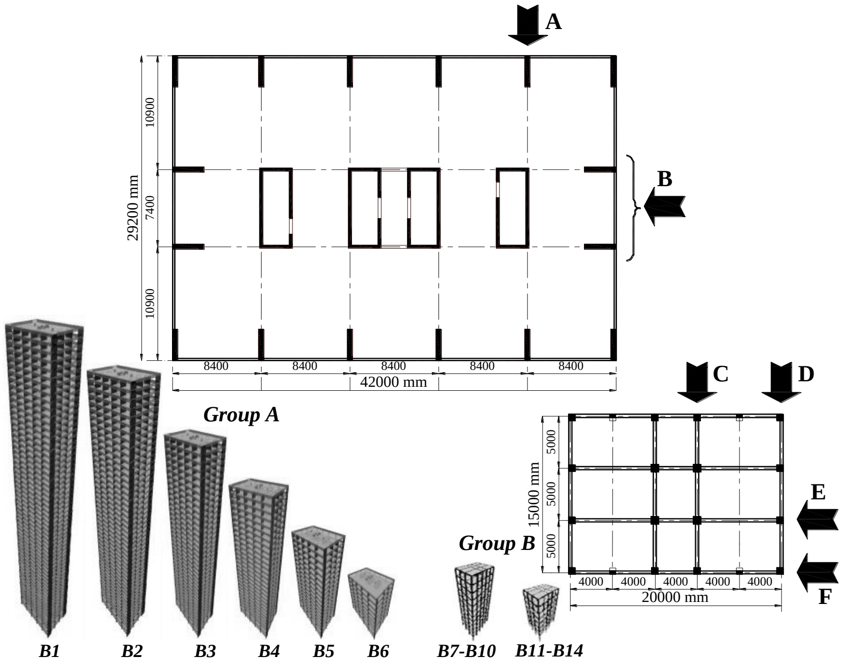

Figure 3. Layouts and finite element models of the fourteen reference structures showing different lateral force-resisting systems in the longitudinal and transverse directions.

Ten natural and synthetically-generated accelerograms are selected for analysis of Group A buildings based on the conclusions of a hazard study for Dubai (Mwafy et al. 2006). The selected records represent long period earthquakes, which is a seismic scenario applicable to the reference region. This group of records also matches the 10% probability of exceedance uniform hazard spectrum recommended in the above-mentioned study and the design code spectrum, as shown in Figure 4(a). The analysis of Group B buildings is performed using another set of natural and artificially-generated records. The selected records were scaled to possess equal velocity spectrum intensity in the period range of the buildings (Mwafy and Elnashai 2001). Figure 4(b) compares between the selected set of ground motions for Group B buildings and the design spectrum at a spectral intensity of 0.15g. The selected two sets of input ground motions were scaled to different intensity levels (i.e. 0.5, 1.0, 1.5 and 2.0 times the design earthquake), which enables the assessment of the structures under increasing levels of seismic hazard.

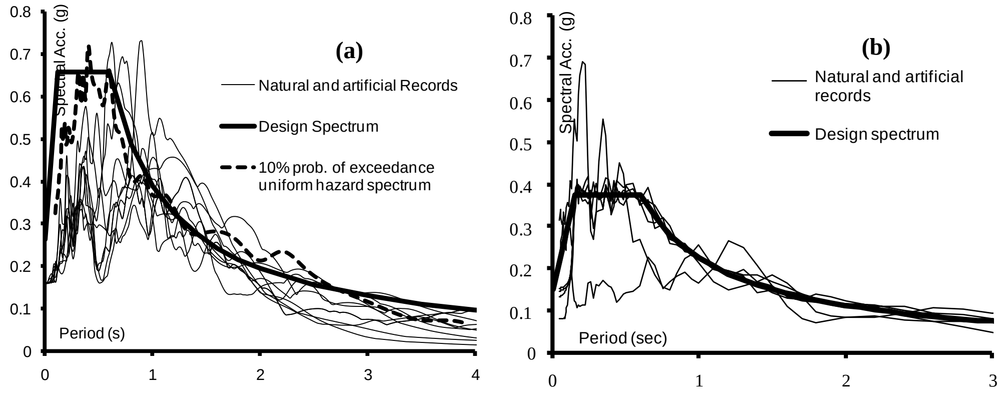

Figure 4. The design spectra along with the response spectra of input ground motions used in analysis of Group A (left, R1-R10) and Group B (right, R11-R16) buildings (5% critical damping).

Although several improvements have been suggested to advance pushover analysis, recently developed procedures still do not guarantee satisfactory results when compared with Incremental Nonlinear Response History Analyses (INRHAs) with increasing the structural irregularity and ground motion peculiarity. Conventional pushover analysis has been employed in previous studies for the capacity estimates of tall buildings (Mwafy et al. 2006). It was suggested to employ the inverted triangular load, which resembles the first mode shape, and uniform load distribution, which represents forces proportional with mass, to conservatively estimate the ultimate capacity of medium- and high-rise buildings, respectively (Mwafy and Elnashai 2001). Research is still needed to improve IPA for the demand prediction of high-rise buildings, maintaining the simplicity which is the most advantageous feature of this technique.

4 CORRELATION OF RESULTS FROM DIFFERENT ANALYSIS PROCEDURES

To correlate the demands from different analysis procedures, IAPs are undertaken using the fiber element models of the reference structures. The selected input ground motions are scaled to different intensity (PGA) levels and response parameters are monitored during the multi-step analyses to measure the level of structural damage. A comparison between the design base shear of Group A buildings calculated using EAPs with the results obtained from 240 NRHAs at different input ground motion intensities are shown in Figure 5. A summary of the median NRHA results from all input ground motions used in analysis is presented in Table 2. For the Group A buildings, the median base shear from NRHA is on average 120% higher than EAPs. This observation is also clear in Group B but with a higher variation between its buildings due to the different PGA and ductility levels used in the design. It is shown that the use of forces obtained from NRHA in design without any reduction is an over-conservative approach. The differences between NRHA and EAPs results are due to the contribution of overstrength, which is only accounted for in NRHA. This issue was also highlighted by FEMA 451 (2006).

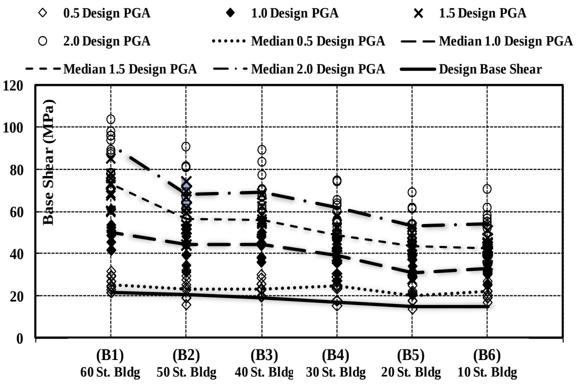

Figure 5. Comparison between the design base shear of Group A buildings and the results obtained from 240 NRHAs at different ground motion intensities.

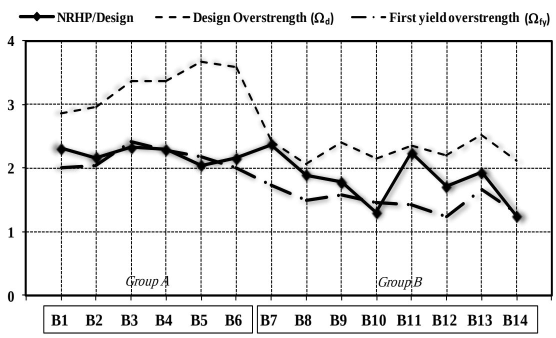

Figure 6. Comparison between the NRHA-to-design strength ratios along with the design and first yield overstrength factors.

The sample IPA results of the reference structres shown in Figure 2 indicate that the design overstrength (Ω_d), defined as the ratio of actual (V_y) to reduced design strength (V_s), is generally high. Figure 2 also shows that the first indication of yielding is significantly higher than the design strength (V_s) for all buildings. The first yield overstrength (Ω_{fy}), which is defined as the ratio of first yielding (V_{fy}) to design strength (V_s), is summarized in Table 2. Figure 6 also compares between the NRHA-to-design strength ratios along with the design and first yield overstrength factors (Ω_d and Ω_{fy}, respectively) of the reference structures. The minimum observed Ω_{fy} factor is 2.0 for Group A and 1.28 for the Group B buildings. This ratio should be as close as possible to unity to meet the modern codes' design philosophy, as shown in Figure 1. It is noteworthy that extensive INRHAs were also carried out to validate the design overstrength (Ω_d) and the first yield overstrength (Ω_y) factors obtained from IPAs. The calculated overstrength factors using the median base shear from INRHA at the collapse limit state were much higher than those obtained from IPA. These differences are due to the limited capability of the latter procedure to predict higher mode effects in the post-elastic range. The IPA results are therefore adopted to conservatively estimate the overstrength factors. Note that the actual overstrength should be higher than the values obtained from both IPA and INRHA due to the contribution of some parameters such as non-structural elements (Elnashai and Mwafy 2002). The presented results suggest that the high force demands obtained from NRHA at the design earthquake compared with elastic analysis procedures are mainly due to the contribution of first yield overstrength (Ω_{fy}). This implies that the first yielding can be safely reduced to the design force level, as recommended in Figure 1, to arrive at consistent and cost-effective design forces. Employing the forces obtained from NRHA without any reduction to refine the design results in larger cross sections and higher reinforcement compared with the initial design.

Table 2. Structural seismic response and overstrength of the reference structures.

		(1)	(2)	(3)	(4)	(5)	(6)	(7)
Group	**Ref.**	**NRHA Base Shear [a] (kN)**	**(1)/V_s**	**Ω_d = V_y/V_s**	**Ω_{fy} = V_{fy}/V_s**	**R = V_e/V_s**	**Ω_i = V_y/V_e = $1/R_\mu$**	**(2)/(4)**
A	B1	50435	2.31	2.86	2.00	4.00	0.72	1.16
	B2	44350	2.16	2.97	2.03	4.00	0.74	1.06
	B3	44260	2.32	3.37	2.42	4.00	0.84	0.96
	B4	39090	2.29	3.38	2.27	4.00	0.85	1.01
	B5	30935	2.04	3.68	2.17	4.00	0.92	0.94
	B6	33200	2.15	3.59	2.00	4.00	0.90	1.08
B	B7	11105	2.37	2.42	1.72	5.00	0.48	1.38
	B8	11316	1.89	2.06	1.48	3.75	0.55	1.28
	B9	5116	1.78	2.40	1.58	3.75	0.64	1.13
	B10	5661	1.30	2.14	1.46	2.50	0.86	0.89
	B11	9007	2.24	2.35	1.42	4.00	0.59	1.58
	B12	9168	1.71	2.20	1.25	3.00	0.73	1.37
	B13	4908	1.93	2.51	1.66	3.00	0.84	1.16
	B14	4722	1.24	2.10	1.28	2.00	1.05	0.97

a: Median demands of input ground motions used in NRHA at the design PGA

5 USE OF FIRST YIELD OVERSTRENGTH IN DESIGN

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). The response is in the elastic range under the design earthquake if Ω_i exceeds unity (uneconomical design), while the lateral behavior is unacceptable if Ω_i is less than 1/R, which implies an ultimate capacity (V_y) less than the design strength (V_s). Therefore, the Ω_i factor directly reflects the anticipated behavior of the structure under the design earthquake. Table 2 shows the average inherent overstrength factor (Ω_i) of the investigated buildings, while Figure 7 depicts a pictorial view of the buildings response measured using Ω_i. The results indicate a possibility to refine the design of buildings B5 and B6 since their seismic response is near the elastic limit (Ω_i = 1). A full elastic response is also anticipated from the calculated Ω_i factor of Building B14. Figure 7 shows the higher reserve strength of the 8-storey irregular buildings (B11-B14) compared to the 12-storey regular structures (B7-B10) due to the conservative R factors adopted by the design code for irregular structures.

Based on the above-mentioned discussion it is suggested to use the first yield overstrength (Ω_{fy}) along with the inherent overstrength factor (Ω_i) for refinement of the initial design. Figure 8 summarizes the proposed approach. Pushover analysis is first conducted to estimate Ω_{fy} and Ω_i. It is recommended to use NRHA to refine the initial design if Ω_i indicates unacceptable or uneconomical response, as explained in Figure 7. To rationally exploit NRHA results in design, a realistic reduction factor of strength demands is proposed. This scaling factor is equivalent to the first yield overstrength (Ω_{fy}). A safety factor (Φ) is recommended to account for uncertainties associated with estimating the inherent overstrength factor. It is clear from the last column in Table 2 that a safety factor (Φ) of 0.89 is adequate. Although the reference structures cover a wide spectrum of buildings, sensitivity analysis may be needed to fully calibrate this safety factor. It is therefore suggested to use a conservative safety factor (Φ) of 0.8 pending its final calibration. The above discussion results in a reduction factor for inelastic analysis $R_{in} = \Phi \,.\, \Omega_{fy}$. It is also recommended to verify the refined design obtained from the abovementioned approach using pushover analysis and the inherent overstrength measure as a final design step. It is worth noting that seismic codes also impose empirical limitations on the reduction of base shear obtained from dynamic analysis procedures (ASCE-7 2010). These limitations are justified by several uncertainties in modeling and analysis (FEMA 2009). The presented theoretically-based approach exploits the actual ductility and overstrength 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 tools. The work presented in the present study is intended to support ongoing activities towards having different structures with comparable reliability and cost effectiveness, an objective not fully achieved by existing code design procedures.

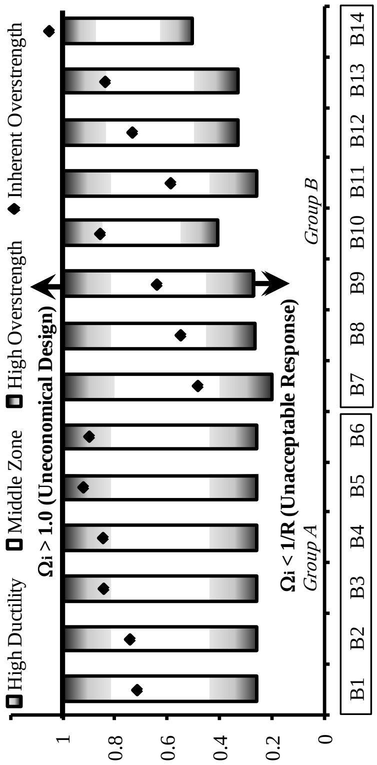

Figure 7. Assessment of response using inherent overstrength ($\Omega_i = V_y/V_e$).

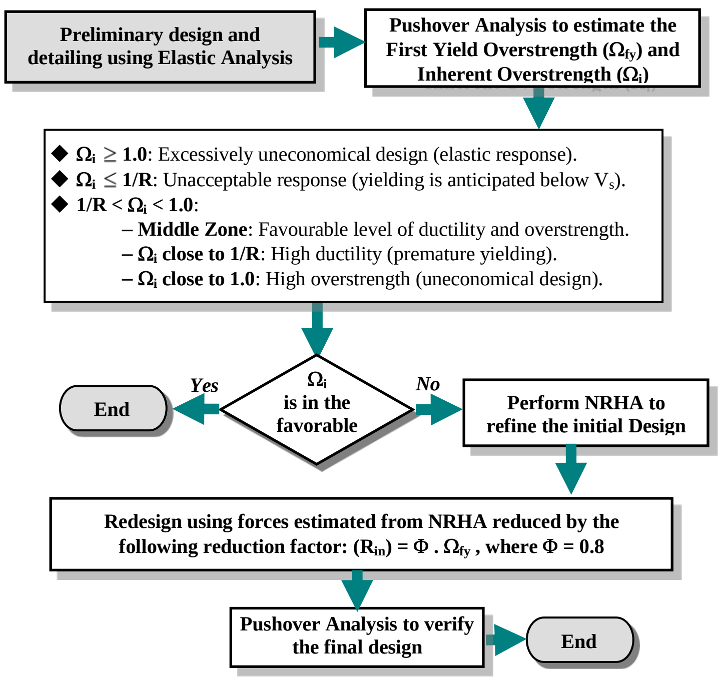

Figure 8. Proposed approach for refining the design using inelastic analysis and first yield overstrength.

6 CONCLUSIONS

Two sets of medium- and high-rise buildings were assessed in this study using verified analysis tools and refined modeling approaches to correlate the results of different elastic and inelastic analysis procedures recommended by modern seismic codes. Realistic input ground motions were adopted based on design spectra and seismicity of the regions from where the reference buildings were selected. The demands obtained from elastic and inelastic analysis procedures confirmed the over-conservative strength demands obtained from inelastic analysis if used for refining the initial design. A simple measure of response, termed the inherent overstrength factor, was employed for evaluating the anticipated behavior of the initial design under the design earthquake. The present study confirmed that the nonlinear response history analysis can be effectively used to refine the initial design of buildings with different structural systems and heights if the response was outside a favorable range. The presented approach exploits inelastic analysis tools, which is believed to play a more influential role in seismic design in future. The approach also effectively utilizes the first yield overstrength, aiming at having different structures with similar reliability and cost effectiveness.

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