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Invited Paper

Fundamental Concepts of Performance Based Earthquake Engineering

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Abstract. In modern earthquake engineering, seismic performance properties are more important than the resistance properties of structures. Seismic performance of a structure is related to the expected damage level under an earthquake excitation, and how this damage relates to safety and use of the building. Accordingly, estimation of member deformations becomes more critical than the estimation of internal forces since the deformations in the post elastic regions of the structure are well correlated with damage in these regions.

Fundamental issues of performance based seismic design in the next generation of seismic codes are evaluated in this paper. Their ranges of applicability, limitations and prediction accuracies are evaluated comparatively. Finally, difficulties in the practical implementation of these procedures and the required level of engineering knowledge are discussed. Basic examples are presented on the implementation of performance based design.

Keywords: Performance based design; Performance Evaluation; Nonlinear procedures; Deformations.

1. Introduction

Performance based earthquake engineering can be defined with general terms as follows:

A structure should display a performance under a specified earthquake excitation as predicted by the engineer, in both deformation distributions and internal force distributions. If damage is permitted, then these damages should occur at designated locations, in specified amounts.

These performance targets also exist in conventional earthquake resistant design, although implicitly and indirectly. However realistic estimation of internal deformations and force distributions, and accordingly accurate control and verification of structural response is not possible by employing classical analysis procedures. Significant progress has been achieved in recent years both in the estimation of seismic hazard, and mechanical modelling of structural materials and structural analysis procedures.

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Seismic hazard can now be expressed in terms of site specific spectral acceleration maps which is a consequence of a rich database of earthquake ground motions compiled with strong motion station networks deployed in the last decades, and recent advances in the ground motion prediction models as well as probabilistic analysis procedures. Accordingly, a site specific design spectrum can be defined for any geographical point on the earth which also reflects the local soil properties. Seismic hazard can be specified for a construction site with significant reliability, for several exceedance probabilities such as 70, 50, 10 and 2 percent in 50 years. In return, earthquake engineers have the opportunity to select different performance levels under different seismic intensities. On the other hand, recent advances in the modeling of nonlinear structural material and component behavior, and advances in the simulation of nonlinear behavior with effective computational procedures paved way to the development of rigorous and realistic structural models. The analysis of reinforced concrete shear walls, which is a source of traditional difficulty in seismic analysis, is a prominent example. With the implementation of fiber modelling techniques to concrete walls, their nonlinear dynamic response can be calculated much fast and easily compared to the nonlinear finite element analysis procedures. Furthermore, analytical models which represent nonlinear behavior of beam-column connections and flexure-shear behavior of concrete frame members can be calibrated with experimental data and implemented in the new generation of structural analysis software. Finally, nonlinear time history analysis procedures and their simpler variants, i.e. multi-mode nonlinear static analysis procedures are being employed commonly as standard tools in earthquake engineering research. When these developments are transmitted into earthquake engineering practice, the actual seismic performance of structures becomes quite “predictable”. This is not possible though with the conventional analysis procedures.

Nonlinear analysis procedures should be employed in order to estimate the seismic performance of structures realistically. Linear elastic procedures are valid within certain limitations, only for estimating the inelastic deformations. Nonlinear procedures are not simple and straight forward however as the linear elastic procedures. Moreover nonlinear structural analysis procedures are not a part of the standard undergraduate curriculum of civil engineering education. Accordingly, implementation of performance based design and assessment procedures to conventional buildings (new buildings with standard height and fixed base) is early at this stage. Their design shall be carried out with conventional procedures also in the first generation of performance based seismic codes. Performance based procedures in the new generation of seismic codes are currently applicable to the seismic design of tall buildings (TBI, 2010; LATBDC, 2014; TEC 2015), design of buildings with seismic isolation (Eurocode 8 Part 1, 2005; ASCE 7-10, 2010, TEC 2015), seismic rehabilitation of existing structures (ASCE 41, 2007, Eurocode 8 Part 3, 2004; TEC 2015) and seismic performance, risk and safety assessment of existing buildings (FEMA P-58-1, 2012). Their application scope will inevitably widen in the near future. For example, consideration of infill walls as structural members in design is one of the foreseeable targets because infill walls has a significant effect on the strength and stiffness of frame structures meanwhile their poor seismic performance usually becomes a dominant factor in determining the overall structural performance, particularly under moderate level ground shaking.

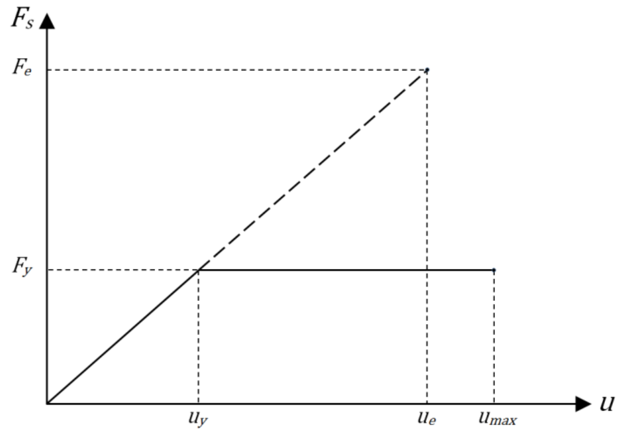
2. Comparative Evaluation of Strength-Based and Deformation-Based Seismic Design Approaches

Estimation of internal forces is the main focus of strength based seismic design whereas the estimation of deformations is the basis of performance based design.

Design forces in current strength based seismic codes are roughly estimated by reducing the forces calculated by linear elastic analysis procedures with a heuristic *force reduction*, or *response reduction factor*. This reduction is based on the premise that the structure shall undergo ductile inelastic deformations under the design earthquake. Ductile inelastic seismic response characteristics are imposed on the system through the implementation of capacity design principles. There are two fundamental rules of capacity design. The first one is the *strong column-weak beam principle* which controls plastic hinging hierarchy by permitting plastic hinge formation first at the beam ends rather than the column ends since beams are not critical from the vertical stability point of view. The second one is the *capacity shear principle* where the design shear is not calculated from analysis, but from the equilibrium of forces in a member with plastic flexural hinges formed at both ends. Capacity shear approach prevents shear failure in structural members. Additional detailing rules are enforced, particularly on the confinement of plastic end regions, in order to provide larger plastic rotation capacities to critical member ends.

Ductility capacity provided by capacity design and the inherent deformation capacity of the system related to material and system characteristics are expressed with a single response reduction factor, R. If the lateral response of the system can be idealized as a single degree of freedom elasto-plastic system as shown in Fig. 1, then R factor represents the ratio of maximum seismic force (F_e) acting on the linear elastic system, to the lateral load capacity (F_y). An inherent assumption here is that the linear elastic and the inelastic systems exhibit approximately the same lateral displacement under the same earthquake ground motion in accordance with the equal displacement principle, i.e. ($u_e \approx u_{max}$).

The basic assumption of representing nonlinear response with linear elastic modeling is never verified under actual seismic forces in conventional strength based design. This is indeed a fundamental weakness. When the flexural capacities provided by strength design under reduced earthquake forces are exceeded, the actual inelastic deformations, and accordingly the actual damage can never be estimated by linear elastic analysis procedures. In other words, strength based design is not *transparent*. The consistency of initial assumptions with the final outcome is never known. The risks which arise due to the differences are usually overcome by a highly conservative design dictated by design codes. However such a strength based design philosophy leads to very uneconomical designs especially for long period structures (tall buildings, base isolated buildings, etc.), hence limits its use. One of the basic objectives of performance based design is to overcome such difficulties which cannot be handled by the standard analysis procedures in current seismic codes.



$$\boxed{R_\mu = F_e / F_y}$$

Fig. 1 Idealized lateral force-deformation characteristics of linear elastic and elasto-plastic equivalent single degree of freedom systems, and response reduction factor

A preliminary design is conducted first in performance based design under service level loads (gravity loads combined with wind loads or ground shaking due to frequent earthquakes). Then structural performance obtained with this design is checked under various levels of earthquake intensities. The expected performances, hence damage distribution under light, moderate and severe ground shaking are expressed explicitly in terms of inelastic deformations and the corresponding performance states. Finally the calculated performances are verified separately under each earthquake intensity. If the performance objectives are not satisfied, design is revised. If the performance objectives cannot be satisfied with conventional structural design solutions and construction practices, new technologies can be employed such as seismic isolation and energy dissipation devices.

The classical strength based design and the new generation of performance based design approaches under seismic effects are evaluated comparatively in the following sections with due considerations of seismic hazard, performance targets, design approach and analysis procedures.

2.1 Seismic Hazard

A single level seismic hazard is defined in strength based design, as the earthquake intensity with a 10% probability of exceedance in 50 years, corresponding to a return period of 475 years. In most current seismic codes including the Eurocode and the Turkish Code, 10% / 50 earthquake intensity is defined with the effective peak ground acceleration given by a seismic zone map shown in Fig. 2 for Turkey and Italy. Then this value is converted into a design spectrum by fitting a standard spectral shape which is anchored to effective peak acceleration at zero period. This standard spectral shape is further modified to represents local soil conditions approximately.

Earthquake design spectra in the new generation of seismic codes are calculated directly from the short period and long period spectral accelerations, expressed in the form of contour maps. Short-period spectral acceleration ordinates (S_{DS} at T=0.2 second) and long-period period spectral acceleration ordinates (S_{D1} at T=1 second) are obtained from the contour maps for the geographical coordinates of the construction site. The contour maps for S_{DS} and S_{D1} in the next seismic code of Turkey are shown in Fig. 3. The design spectrum is then constructed with a procedure which takes into account local soil conditions (Fig. 3.c).

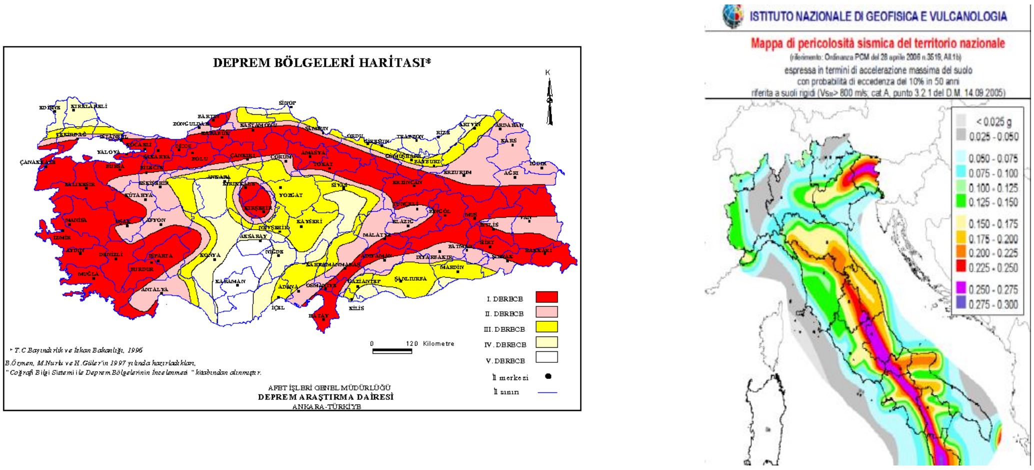
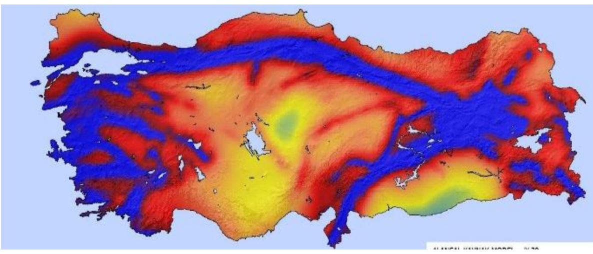
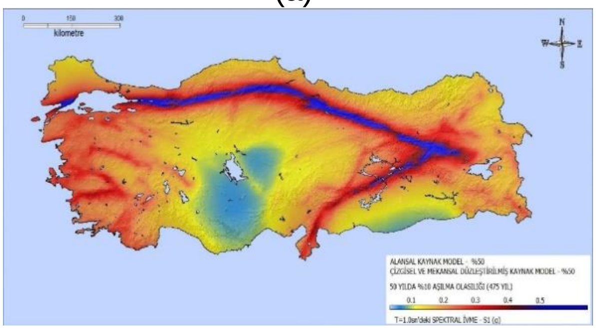


Fig. 2 Seismic zone maps of Turkey and Italy for effective peak ground acceleration with 10% probability of exceedance in 50 years



(a)



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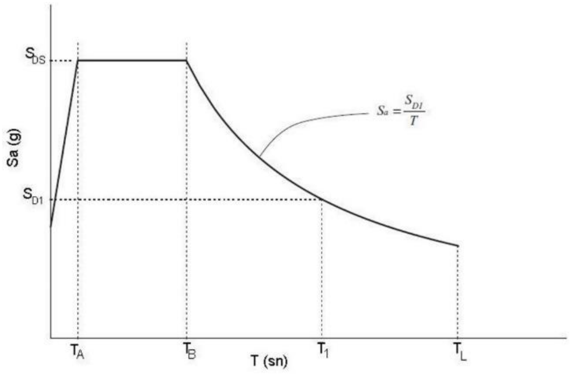

(c)

Fig. 3 Spectral acceleration maps for Turkey. (a) S_{DS} map at T= 0.2 s, (b) S_{D1} map at T=1.0 s, (c) linear elastic design spectrum for stiff soil sites. All values are given for 10% probability of exceedance in 50 years

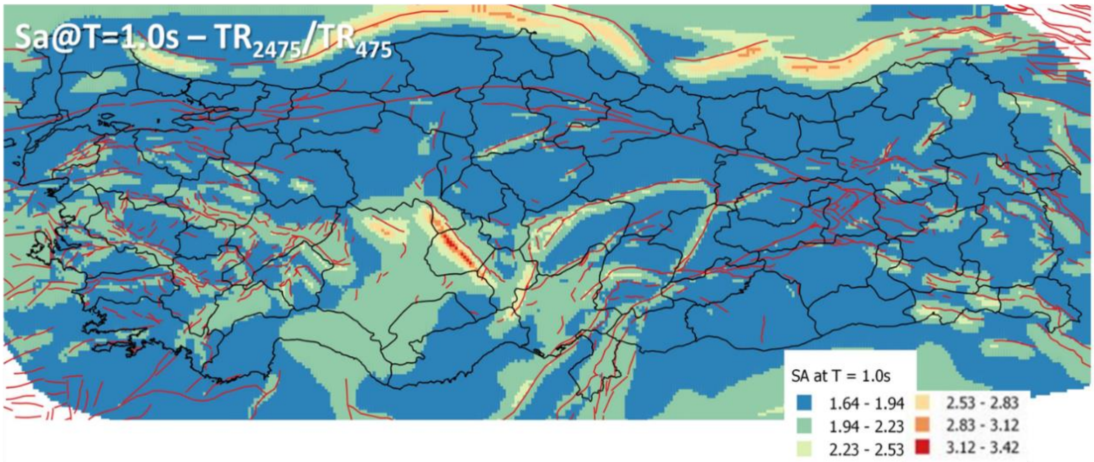

Fig. 4 The ratio of 2475 to 475 year spectral accelerations at T=1 second in the new Turkish Spectral Acceleration Map.

There is an important difference between the design spectra of strength based and performance based design approaches. The falling branch of the spectrum in strength based design follows ($1/T^{0.8}$) or ($1/T^{2/3}$) curves whereas it is represented by the more realistic ($1/T$) curve in Fig. 3.c above. ($1/T^{0.8}$) gives higher values compared to ($1/T$) at longer periods since the objective in strength based design is to provide an extra safety for flexible structures by increasing their stiffness and strength in order to reduce displacements. However this conservatism creates a major handicap in the design of tall buildings and base isolated buildings where the fundamental periods are usually longer than 2 seconds.

Different seismic intensities are expressed in terms of different probabilities of exceedance of spectral accelerations in performance based seismic design. These intensities are generally defined by 70% or 50% probability of exceedance in 50 years (43 and 72 years, respectively) for frequent earthquakes, 10% probability of exceedance in 50 years (475 years) for strong earthquakes and 2% probability of exceedance in 50 years (2475 years) for the maximum earthquake expected at the site. The probability of exceeding spectral accelerations of the design spectrum are equal for all period values, i.e. they constitute a uniform hazard spectrum (Sucuoğlu and Akkar, 2014).

Different seismic hazard maps for S_{DS} and S_{D1} are not yet available for different probabilities of exceedance, or return periods in the new generation of seismic codes. They can be obtained from each other practically by using scale factors. For example, the spectra in Fig. 3 can be scaled by 1.5 in order to obtain the 2475 year spectra. The factor 1.5 is in fact the importance factor I in the strength based design codes. Accordingly, the same life safety performance which is expected from an ordinary building under the 475 year hazard is expected from an important building (I=1.5) under the 2475 year earthquake. The NEHRP (2009) specifications in USA does the opposite, where the seismic hazard maps are only provided for 2475 year hazard level and design spectrum for 475 year hazard is obtained by using a scale factor of 2/3. These constant scaling factors do not reflect reality so well in all seismic regions. For example, the ratio of 2475/475 year spectral accelerations at one second period are presented in

Fig. 4 for Turkey. It can be observed that this ratio is usually larger than 1.5, especially in the lower seismicity regions. Therefore scaling the 475 year design spectrum by 1.5 may be unsafe in the safety verification of important buildings under the maximum, or 2475 year earthquake.

2.2 Performance Targets

An explicit, quantitative description of performance targets is not possible in strength based design. They can only be defined qualitatively. The performance targets in strength-based design codes are usually defined as follows: *No damage under low intensity ground shaking, limited damage under moderate level shaking and residual damage, but life safety under strong ground shaking*. It is not clear however what the low, moderate and strong ground shaking intensities are. Design earthquake is generally accepted to represent a strong earthquake intensity, and if life safety or "no collapse" target is satisfied under design earthquake, it is assumed that the other higher performance targets under lower intensities are satisfied automatically. But these are never verified. Moreover, these performance targets are not quantifiable since they cannot be measured through linear elastic analysis procedures, but may only be verified by site investigations after an earthquake which produced design level ground motions over the affected region.

Seismic performance targets are explicitly quantified at the member or section level in performance based earthquake engineering, and verified by appropriate analysis at every performance level. Accordingly, performance based earthquake engineering offers a *transparent* design methodology. A comparative evaluation of the performance targets of these two design approaches is introduced on the moment-curvature diagram of a typical reinforced concrete member cross section with the aid of Fig. 5.

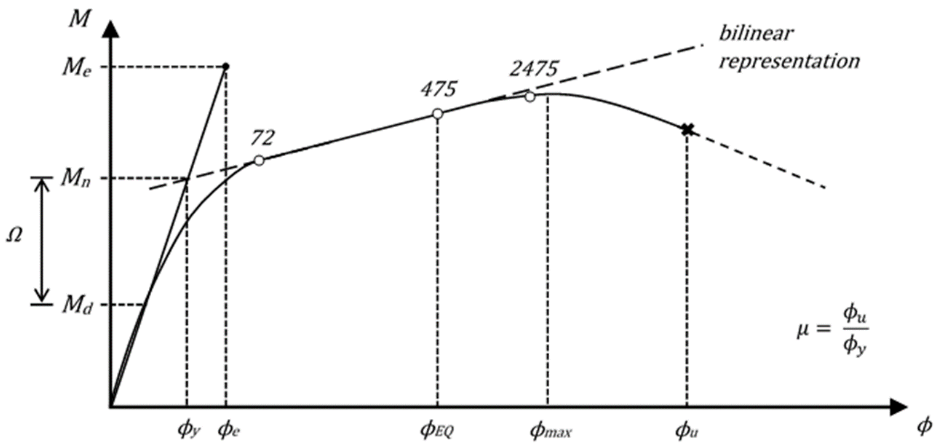

Fig. 5 Moment-curvature envelope relationship of a typical reinforced concrete section. M_d is the design moment, M_n is the nominal flexural strength, M_e is the flexural demand from linear elastic analysis, ϕ_y is the yield curvature, ϕ_u is the ultimate curvature capacity.

There is only a strength check in strength based design:

$$M_n > M_d = \frac{M_e}{R} \qquad ; R = \frac{M_e}{M_d} \tag{1}$$

Here, R is the response (load) reduction factor which is composed of two parts:

$$R = R_\mu \times R_\Omega \;\; ; \;\; R_\mu = \frac{M_e}{M_n} \;\; ; \;\; R_\Omega = \frac{M_n}{M_d} \tag{2}$$

R_μ is the ductility reduction factor and it is closely related with the ductility ratio μ through the well-known R- μ -T relationship in earthquake engineering. R_Ω on the other hand is the overstrength factor. Ω represents overstrength in strength based design which is generated without intention. The section is actually designed for M_d, and the nominal capacity develops with an overstrength which results from minimum dimensions, minimum reinforcement, load and material factors, other load combinations, etc. Design is deemed satisfactory if the nominal, or existing strength exceeds the strength demand through Eq. (1). However the actual curvature attained under earthquake loads (ϕ_{EQ}) is never calculated and it is not known to which performance does this curvature or deformation corresponds to. Hence a performance verification is not accomplished.

Deformations are evaluated rather than the forces in performance based design. Performance level of the cross section is determined under a given hazard level, say 475 year earthquake, by comparing the deformation demand ϕ_{EQ} with the associated performance limit in view of Fig. 5.

$\phi_{EQ} < \phi_y$: No damage (3.a)
$\phi_y < \phi_{EQ} < 0.75\ \phi_{max}$: Limited damage (Life safety performance) (3.b)
$0.75\ \phi_{max} < \phi_{EQ} < \phi_{max}$: Stability (No collapse performance) (3.c)

Either curvatures or member end rotations can be employed as the performance parameter in performance evaluation. Plastic curvature demand (ϕ_{EQ} - ϕ_y) can be converted into plastic end rotation demand by introducing the plastic hinge length L_p of the member. L_p can be taken as half of the member depth for prismatic concrete sections.

$$\theta_{p,EQ} = (\phi_{EQ} - \phi_y) L_p \tag{4}$$

Performance limits are defined in terms of plastic rotations in Eurocode 8 and ASCE 41. Plastic rotation limits are specified by employing the data obtained from experimental observations. These limits are functions of axial load, shear force, type of reinforcement and detailing in reinforced concrete members.

It is also possible to employ material strains as well in performance evaluation of a plane section shown in Fig. 6 by using the strain-curvature relations. Concrete strains at the extreme fibers of the unconfined and confined regions, as well as the steel strains at the outermost longitudinal reinforcing bars can be calculated from the first principles of mechanics by using “plane sections remain plane” assumption under flexure.

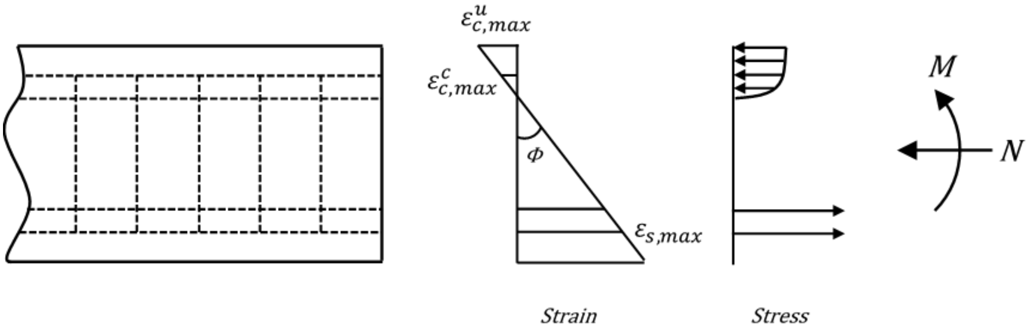

Fig. 6 The basic relations between the curvature and material fiber strains of a concrete section under bending

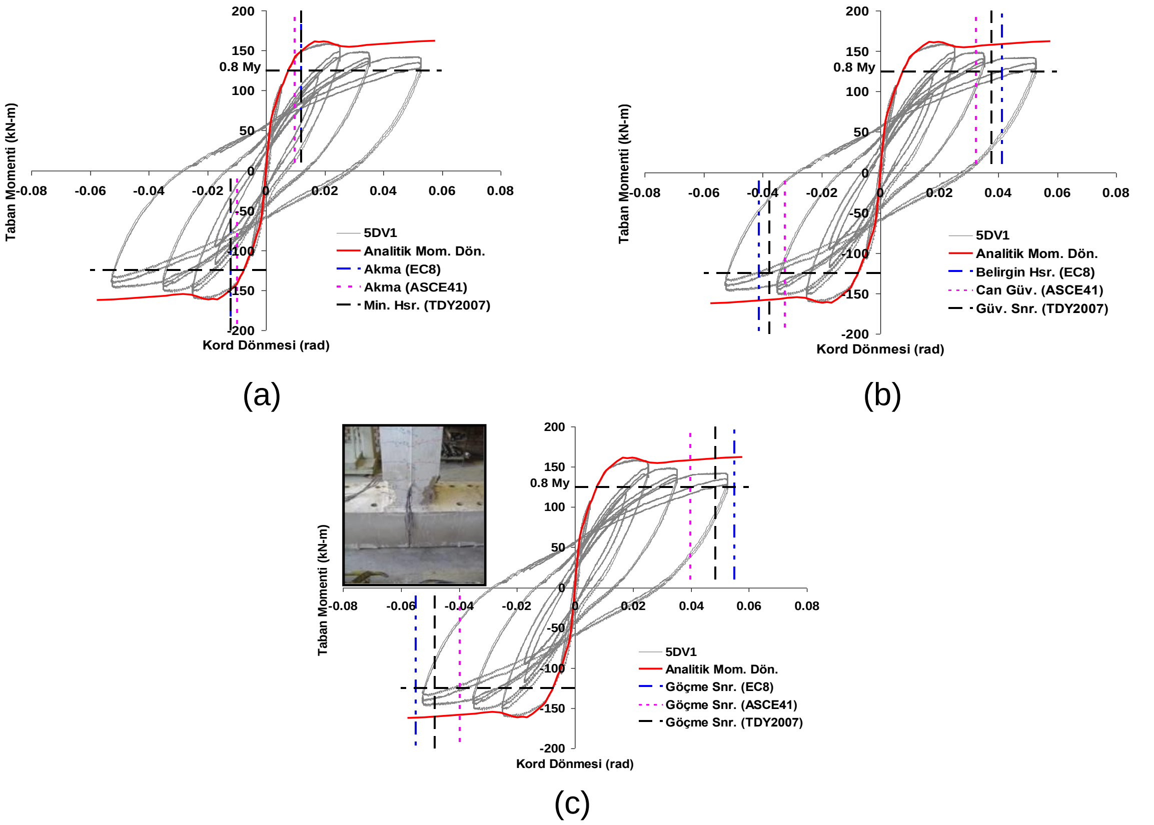

Fig. 7. Comparison of performance limit states given in ASCE 41, Eurocode 8 and TEC 2007 for a code-designed reinforced concrete column under increasing load reversals. (a) Minimum damage or yield, (b) life safety or significant damage, (c) collapse prevention or ultimate state performance limits.

Material performance limits can be specified by considering their yield and rupture strains. This is the approach in the Turkish Earthquake Code (2007), given in Eq. (5) for minimum damage (*MN*), life safety (*LS*) and collapse prevention (*CP*) limit states.

$$(\varepsilon_c^u)_{MN} = 0.0035\,;\ (\varepsilon_s)_{MN} = 0.010 \tag{5.a}$$

$$(\varepsilon_c^c)_{LS} = 0.0035 + 0.01\left({}^{\rho_s}/_{\rho_{sm}}\right) < 0.0135;\ (\varepsilon_s)_{LS} = 0.040 \tag{5.b}$$

$$(\varepsilon_c^c)_{CP} = 0.004 + 0.014\left({}^{\rho_s}/_{\rho_{sm}}\right) < 0.018\ ;\ (\varepsilon_s)_{CP} = 0.060 \tag{5.c}$$

Perfect flexure behaviour and full bond between steel and concrete are inherently assumed in deriving Eqs. (5). When reinforcing bars are not deformed and shear-flexure interaction is significant, reductions can be applied to Eqs. (5) in accordance with experimental observations.

Moment-end rotation relationship obtained at the base of a code-designed cantilever column under lateral load reversals with increasing amplitudes are shown in Fig. 7. Measured column performance is compared with the ASCE 41, Eurocode 8 and TEC 2007 performance limits for minimum damage or yield, significant damage or life safety, and collapse prevention or ultimate limit states (Acun and Sucuoğlu, 2011). Strength degradation is considered insignificant unless the peak strength in a cycle falls below $0.8M_y$. If it falls, the section is considered to lose its capacity, i.e. it fails. Strain limits defined in Eqs. (5) are converted into total member end rotations with the aid of Eq. (4) and Fig. 6. Chord rotation and end rotation at the base are identical in cantilever columns. It can be observed from Fig. 7 (a) that yield or minimum damage limits of the three codes are quite similar. Life safety or significant damage limits shown in Fig. 7 (b) are also consistent. There are some differences for collapse prevention or ultimate performance limits in Fig. 7 (c), however the differences are not remarkable. Therefore selection of the member performance parameter, whether plastic strain, plastic curvature or plastic end rotation, does not matter much unless the results are consistent with experimental observations.

2.3 Design Approach and Analysis Procedures

In strength based design, a response reduction factor *R* is selected first by assuming inelastic response under the design earthquake with 10% probability of exceeding in 50 years (475 year return period), as indicated previously. Then internal forces calculated by a linear elastic analysis procedure (equivalent lateral load or mode superposition) under design earthquake loads are divided by the *R* factor in order to obtain the (reduced) design forces. Structural members are finally designed under these forces and their strengths are assigned accordingly, in line with Eq. (1) which satisfies the (strength > internal force demand) inequality.

The *R* factor in the first step is selected with the assumption of a ductile inelastic system response. Capacity design principles and special seismic detailing requirements are implemented in design for providing the assumed ductility capacity to structural system. However deformation capacity of the system is never checked against the actual deformation demands of the design earthquake. In other words, the expected performance of the system is not verified after completing the design. Only interstory drifts calculated under linear elastic (unreduced) earthquake forces by equal displacement assumption are controlled for protecting the non-structural components attached to the frame.

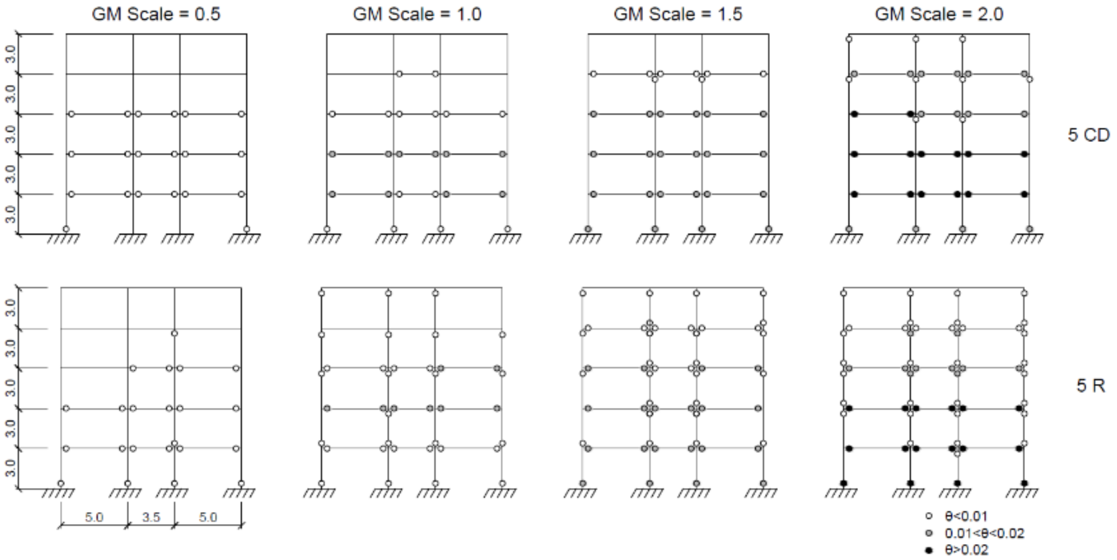

Fig. 8. Seismic performances of Frame 5CD with full capacity design, and Frame 5R with relaxed capacity design. Circular dots indicate plastic rotations, with legend at the inset.

The design procedure outlined here is not transparent as far as the expected seismic performances are considered. A realistic performance assessment is possible only by conducting nonlinear dynamic analysis under different seismic hazard levels representing frequent, moderate and severe seismic actions. However design forces are calculated for a single performance level in strength based design, with linear elastic procedures which are not capable of simulating inelastic response.

Performance-based seismic design starts with a preliminary design under gravity loads and service level earthquake forces, with exceedance probability of 50 or 70% in 50 years (72 or 43 year earthquake hazard, respectively). Then limited damage performance is verified under the 475 year design earthquake, and no collapse performance is verified under the 2475 year maximum earthquake. These verifications are carried out by employing nonlinear dynamic analysis procedures and under realistic earthquake ground motions. The actual, or expected material strengths rather than the design strengths are used in nonlinear structural models. Target structural performance is satisfied if the deformation demands of the critical structural components remain below the deformation limits associated with the target performances. If the desired performance is not achieved, or the plastic deformations are larger than the deformation limits, structural design is revised.

When seismic design requirements in the current earthquake codes are completely satisfied in the design of ordinary structures, strength based design usually achieves the desired seismic performance. Two similar 5 story frames with different levels of capacity design are shown in Fig. 8. Full capacity design is implemented in frame 5CD whereas column reinforcement ratios are reduced in frame 5R. Column-to-beam capacity ratios for both frames are given in Table 1. Modal vibration periods for the first three modes are 1.13, 0.36 and 0.20 seconds, respectively. A response reduction factor
Table 1 Column-to-Beam Flexural Capacity Ratios at Joints

Story #	Exterior Joints	Interior Joints	Exterior Joints	Interior Joints
	5CD		**5R**	
1	2,84	1,59	2,25	1,27
2	2,68	1,48	2,08	1,17
3	2,46	1,34	1,91	1,05
4	2,25	1,19	1,73	0,91

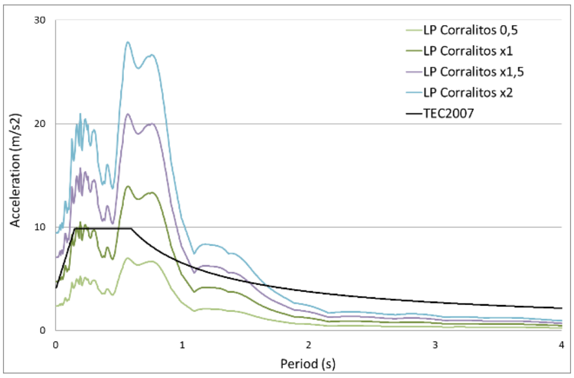

Fig. 9. Acceleration spectra of the scaled ground motions and linear elastic design spectrum

of R=8 is used in design. The designed structures have been analyzed by nonlinear time history analysis under 1985 Loma Prieta, Corralitos 090 ground motion record, scaled with 0.5, 1.0, 1.5 and 2.0. Their acceleration spectra are shown in Fig. 9 along with the linear elastic (unreduced) design spectrum. Unscaled Corralitos ground motion matches the design spectrum quite well in the T_1-T_3 range. Maximum plastic rotations measured at member ends are marked on the frames in Fig. 8. It can be observed that frame 5CD with full capacity design satisfies the strong column-weak beam condition under the first three ground motion intensities. Only small plastic rotations (slight damage) occur in columns under GM scale 2.0, however a collapse mechanism does not develop. On the other hand, although the columns are still stronger than the beams in frame 5R with relaxed capacity design, plastic deformations in columns start earlier and a complete collapse mechanism develops under GM scales 1.5 and 2.0. Such a detailed performance assessment can only be realized with a performance based approach.

Nonlinear structural analysis is essential in performance based earthquake engineering. Nonlinear analysis procedures are mainly nonlinear time history analysis and the multi-mode nonlinear static analysis procedures. Nonlinear static (pushover) analysis procedures are much simpler compared to time history analysis considering convergence problems and compilation of the huge output data. Their basic weaknesses are the representation of instantaneous lateral load profiles which continuously changes with spreading nonlinearity during seismic response, and estimation of target displacements. The second one is more important than the first one. On the other hand, linear elastic analysis procedures may estimate inelastic deformations roughly within the extent of equal displacement rule for SDOF systems, however they have no capacity for calculating internal forces during inelastic response. The 12 story frame shown in Fig. 10 is designed for the design spectrum given in Fig. 9, with a response reduction factor *R*=8 and in full compliance with the capacity design principles specified in the current design codes. Then the designed frame is analyzed with nonlinear time history analysis, multi-mode pushover analysis and linear elastic mode superposition analysis (with cracked section rigidities) under the design spectrum compatible ground motions shown in Fig. 10. Interstory drift ratios and mean beam end chord rotations at each story are presented comparatively in Fig. 11. It can be clearly observed that the results of the two nonlinear procedures are quite consistent whereas the results from linear elastic analysis deviate significantly from nonlinear solutions, especially at the higher stories. There is no point to compare internal forces of course.

3. Performance-Based Seismic Design, a Case Study: 34 Story Tall Building

A tall reinforced concrete building with 115 meter free standing height is designed in a severe seismic zone by following a performance based seismic design approach. Typical floor plan and 3D simulation view are shown in Fig. 12. Characteristic concrete strength is 40 MPa.

A preliminary design is carried out by using *R*=6 in order to determine section dimensions and reinforcing details. Service level performance is verified by conducting nonlinear time history analysis under 7 pairs of ground motions representing the seismic hazard with a 43 year return period (70% probability of exceeding in 50 years). Their response spectra are shown in Fig. 13. 2.5% damping is used in time history analysis.

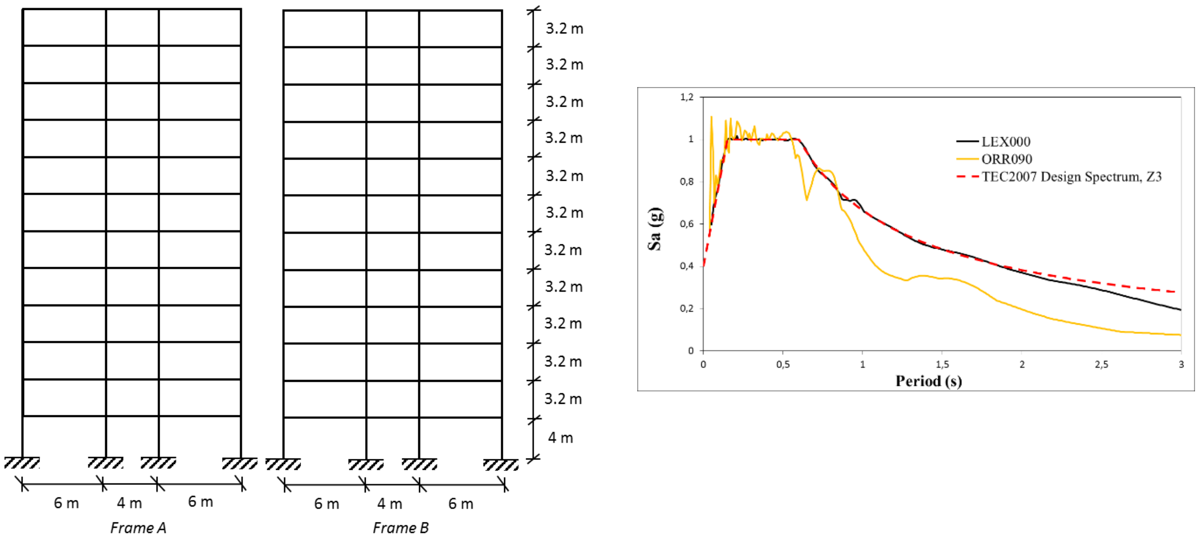

Fig. 10 12 story frame, design spectrum and response spectra of spectrum compatible ground motions

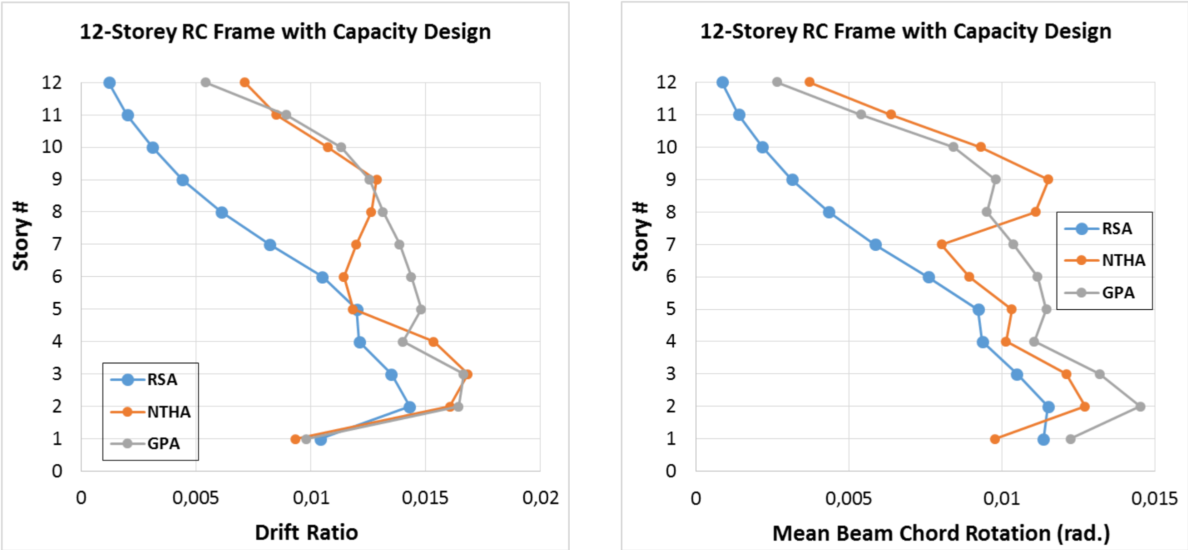

Fig. 11 Comparison of interstory drift ratios and beam-end chord rotations obtained from nonlinear time history analysis (NTHA), generalized pushover analysis (GPA) and linear elastic response spectrum analysis (RSA) with cracked section rigidities, under spectrum compatible ground motions.

Basic performance objectives at the service level are;

a) Mean interstory drift ratio < 0.5 %
b) Member deformations remain below the yield level (no visible damage).

Selected results are presented in Fig. 14. The building satisfies the service performance objectives under gravity loads and the 43 year level earthquake. Drift limit controls performance. When the 0.5% limit is satisfied, all member ends remain well below the yield level in general.

Then similar analyses are repeated under 7 pairs of ground motions representing the maximum earthquake with 2475 years return period. Response spectrum of the ground motions are compatible with the 2475 year uniform hazard spectrum with 2.5% damping. Basic performance objectives at the maximum level (2475 year earthquake) are;

c) Mean Interstory drift ratio < 3.0 %
d) Interstory drift ratio under any earthquake < 4.5 %
e) Residual interstory drift < 1.0 %
f) Column axial loads < 0.4 A_g ($f_{c,exp}$)
g) Shear wall axial stresses < 0.25 $f_{c,exp}$

The expected concrete strength $f_{c,exp}$ is taken as 1.30 times the characteristic strength.

Axial stresses are critical for stability performance under maximum earthquake. Strength degradation is not considered in the current section and material models. Therefore unstable deformation patterns under large displacement reversals cannot be represented in nonlinear analysis. Accordingly, limiting axial stresses serves as a safety guard for preventing collapse.

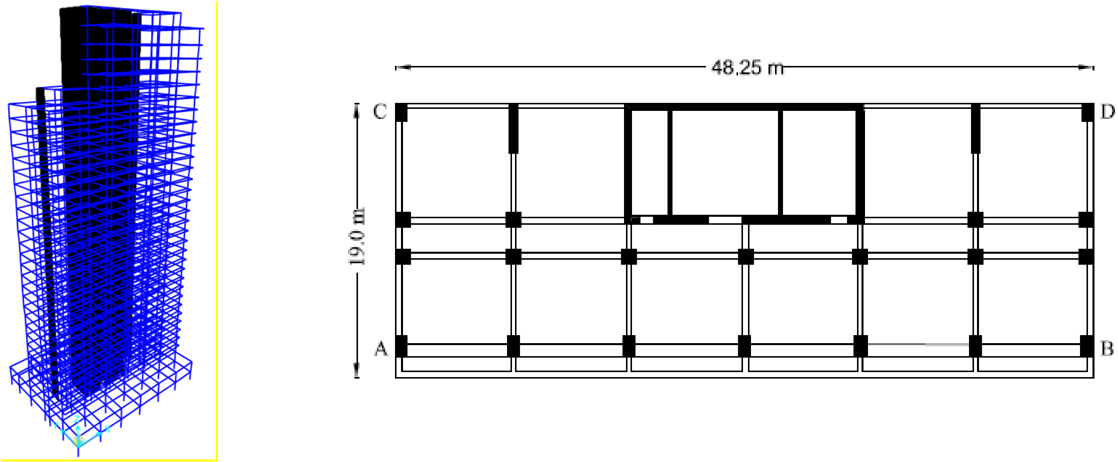

Fig. 12 Isometric view and typical floor plan of the 34 story reinforced concrete tall building

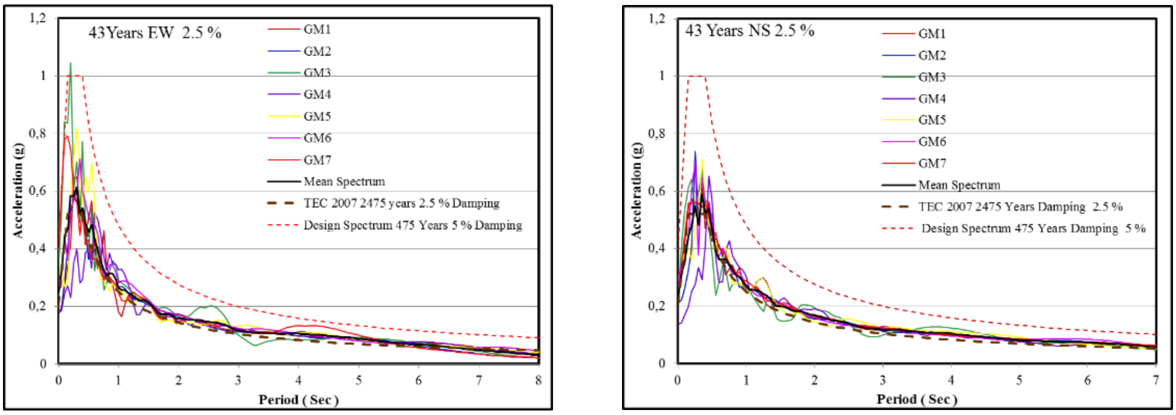

Fig. 13. 2.5% damped response spectra of the 43 year spectrum compatible ground motions and 5% damped design spectrum

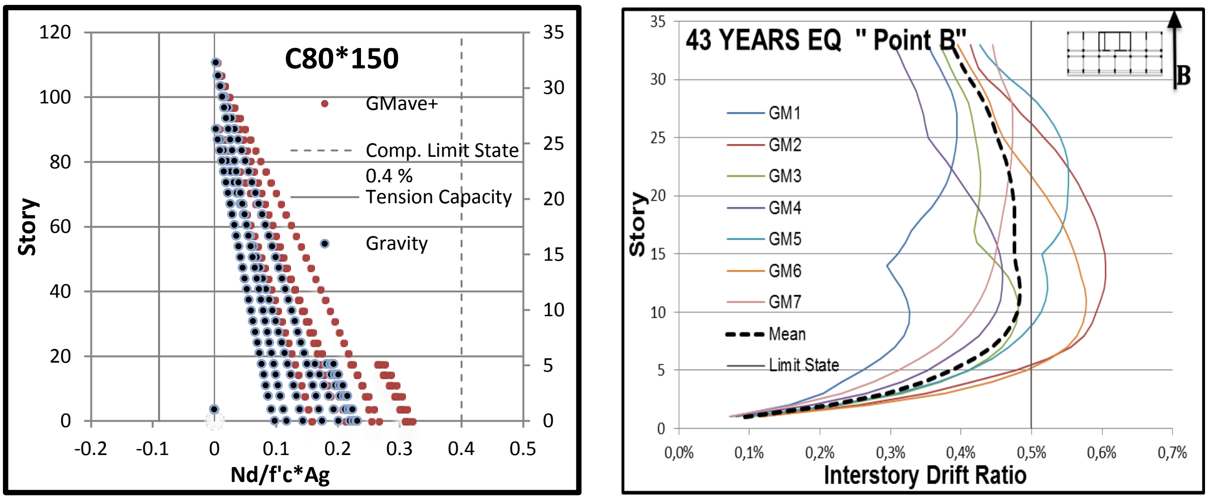

Fig. 14 Column axial loads and interstory drift ratios under 43 year earthquake ground

motions

The results are presented in Figs. 15 and 16. Axial loads in the exterior columns (C80*150) satisfy the axial load limits meanwhile interstory drift ratios satisfy the required performance limits comfortably in Fig. 15. However axial forces in the interior columns (C100*100) and at the bottom stories of the shear wall exceed the specified limits in Fig. 16. Accordingly, reinforcement is increased in these columns at the bottom 5 stories. Column dimensions for the interior columns have been increased from 100*100 cm to 120*120 cm at the first three stories whereas they are extended to the first twelve stories for the three interior columns next to the shear wall. Thickness of the shear wall has been increased from 50 cm to 70 cm throughout the building height and the width of the coupling beams have been also increased from 50 to 70 cm. With these revisions, the building satisfied the performance limits for the maximum earthquake.

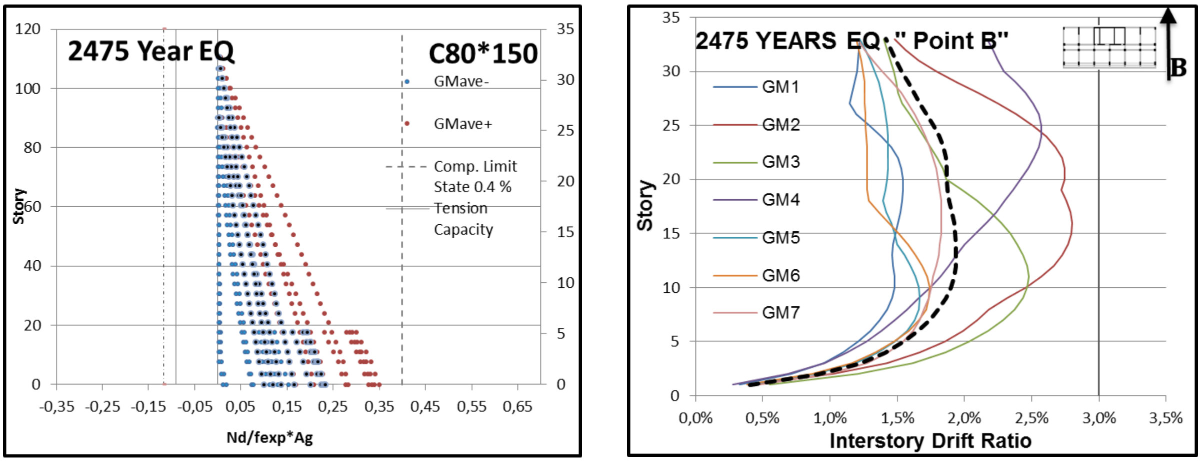

Fig. 15 Column axial loads and interstory drift ratios under 2475 year earthquake ground motions

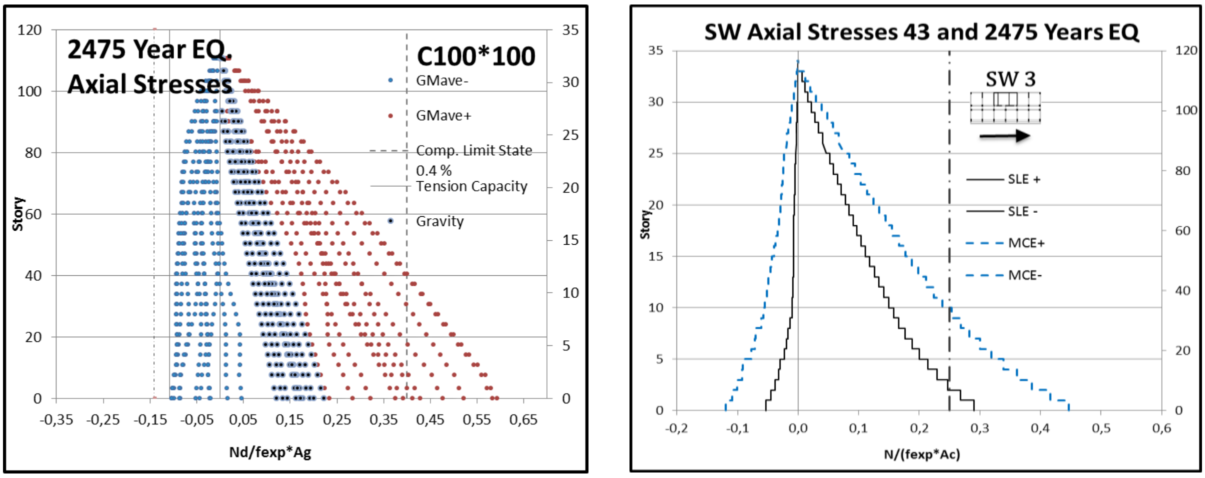

Fig. 16 Column axial loads and shear wall axial stresses under 2475 year earthquake ground motions

4. Conclusions

Performance based earthquake engineering offers a much transparent procedure compared to conventional strength based procedures in the earthquake resistant design of structures.

The nonlinear analysis procedures employed in performance analysis require a higher level of engineering knowledge level compared to standard, conventional engineering practice in both structural modeling, description of inelastic member properties and compilation and interpretation of the analysis results.

Performance based procedures are currently more suitable for the seismic design of critical and long period structures including tall buildings, base isolated buildings and important emergency facilities such as big hospitals, and in the seismic assessment and rehabilitation of important buildings such as hospitals and heritage buildings.

Performance based earthquake engineering will certainly create a demand for better educated earthquake engineers. Implementation of performance based procedures to ordinary buildings will perhaps increase in the future, with further advancement in engineering education and practice.

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