IS THERE DISARRAY IN DESCRIPTION OF PERFORMANCE REQUIREMENTS?

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ABSTRACT: A variety of seismic performance evaluation procedures have been developed for determining the expected performance of reinforced concrete buildings under the effects of prescribed earthquake motions. These procedures are based on linear or nonlinear analyses of the structural models to compute deformation demands for elements. These demands are then compared with performance level-based acceptability limits. This study aims to investigate the correctness of existing assessment procedures using data collected from an actual structure tested in the laboratory. The procedures outlined in FEMA-356, EUROCODE-8 and Turkish Earthquake Code are applied to a full-size, three-story, non-symmetric reinforced concrete building tested at the ELSA laboratory at JRC/Ispra under the SPEAR project. For this purpose, a 3D analytical model of the building is subjected to the records used in the experimental phase and deformation demands are computed according to the procedures described in the guidelines that are being assessed for their correctness. The performance of the structure is evaluated at member level and the accuracy of the considered procedures is rated through comparisons with measurements and observations made after the experiments. The study indicates that the main difference between the procedures stem from different performance-based limit values and the characterizing phrases that are used to qualify them. It appears necessary that a harmonization should be agreed upon before universal application of these procedures. Otherwise the conflicting acceptability criteria among different procedures are likely to create confusion among engineers.

KEYWORDS: 3d analytical model, deformation demand, assessment procedures, performance-based acceptability limits, codes

1. INTRODUCTION

During the last decade, performance-based evaluation has become a fashionable way of predicting the seismic capacities of existing reinforced concrete structures. Many procedures have been proposed in guidelines (and occasionally codes) to estimate deformation demands. These procedures are mostly based on single-degree-of-freedom (SDOF) models with a number of simplifying assumptions. For this reason, there is a need for investigating the adequacy of existing procedures by comparing their results with the measurements of experimental studies on multi-degree-of-freedom (MDOF) systems. In this context, the experimental research on SPEAR test building has been a useful tool to examine the seismic demands of a 3D asymmetric-torsionally unbalanced full-size structure [1].

In this study, the demand values of the analytical model subjected to bi-directional seismic effects (0.15g PGA) are determined and compared with the damage pattern identified through the visual inspections after the test. Our principal aim is to investigate the “correctness” of acceptance criteria stated in Eurocode-8 [2], FEMA-356 [3] and the Turkish Earthquake Code [4] by employing fully nonlinear dynamic analysis procedures as the governing yardstick.

2. DESCRIPTION OF THE TEST BUILDING

A full scale three-storey reinforced concrete frame test building referenced as SPEAR was constructed and tested at the ELSA Laboratory using Pseudo Dynamic (PsD) testing [1]. The building was subjected to both low-level (0.02g PGA) and high-level (0.15g and 0.20g PGA) excitations as its member internal forces and deformations were recorded and natural vibration periods were identified.

The SPEAR building represents older construction in Southern European Countries, such as Greece, without specific provisions for earthquake resistance. The building is regular in elevation; however, the plan configuration is asymmetric about both of the orthogonal axes (Fig. 1). Further details concerning the construction of the SPEAR test building, testing procedure, the mechanical characteristics of the materials and the amount of reinforcement can be found in Molina et al. [1], Jeong and Elnashai [5], Mola et al. [6] and Negro et al. [7].

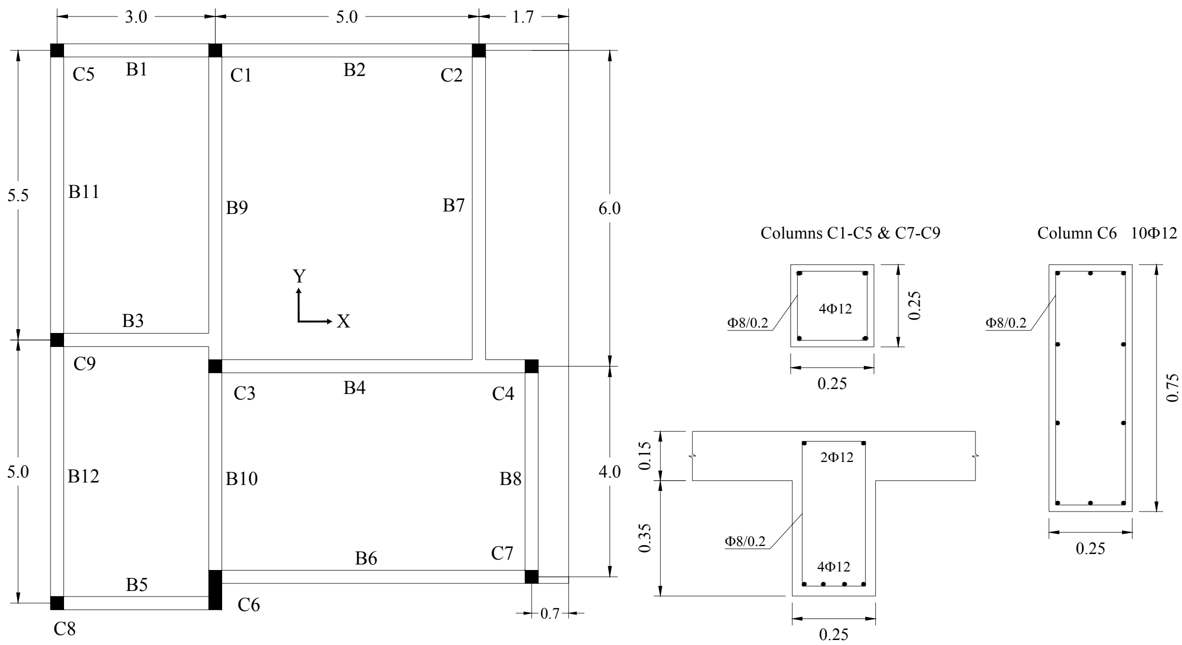

Figure 1 Plan of the test model and drawings of member sections

3. DESCRIPTION OF THE ANALYTICAL MODEL

Nonlinear dynamic analyses of SPEAR test building were carried out with the commercially available software Perform 3D [8]. For modeling reinforced concrete buildings, distributed plasticity is utilized through fiber analysis approach. When modeling, structural dimensions were considered through the section centerlines, masses were assumed as concentrated at the mass centers of each floor, rigid end offsets were not taken into account, but P-delta effects were considered. Following customary practice, T sections were utilized for beam sections and the effective flange width was assumed to be the beam width plus 7% of the clear span of the beam on both side of the web [5]. Damping was not considered in the analytical model as implemented in the PsD test. Other assumptions about material and loading employed in the analytical modeling are summarized in Table 1.

Table 1 Assumptions for analytical modelling

Material	Reinforcement Steel	E_s = 206000 MPa
	Concrete	f_{ck} = 25 MPa E_c = 23750 MPa
	Stress-strain relationship	Reinforcement Steel: Bilinear Model Concrete : Mander Model (Unconfined)
Loading	Gravity Loads	DL + 0.3LL
	Seismic dead load for mass calculation	DL + 0.3LL
	Mass Distribution	Concentrated at the mass centers
	p-delta effect	Not considered
	Centre of mass	Floor 1&2 X=4.58 m Y=5.35 m Floor 3 X=4.65 m Y=5.44 m
	Mass	Floor 1&2 67.26 t Roof 62.08 t
	Mass moment of inertia	Floor 1&2 1500 tm^2 Roof 1363 tm^2

3.1 Verification of the Analytical Model

In order to validate the analytical model employed, top story displacements (Fig. 2) under bi-directional excitation (0.15g PGA) are calculated for both orthogonal directions. The test results of the maximum top story displacements in X and Y directions are compared with the results of analytical model in Table 2. It is observed that the model is quite successful in predicting the test results.

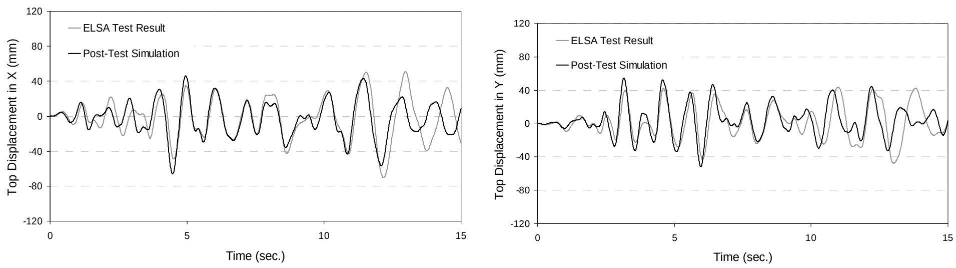

Figure 2 Top story displacements in X and Y directions

Table 2 Comparison of analytical results with the experiment

	Max. Top Displacement (mm)	
	X	**Y**
Test Result	70.06	47.52
Post-Test Simulation Result	66.29	54.08
% difference	-5.38	13.80

4. APPLICATION OF THE ASSESSMENT PROCEDURES TO THE SPEAR BUILDING

Nonlinear Dynamic Procedures are applied to the SPEAR building as defined in Eurocode-8 [2], FEMA-356 [3] and TEC-2007 [4]. Deformation demands are calculated and compared according to the specified acceptance criteria values for all performance levels. Different deformation parameters have been proposed in the prescribed guidelines for checking acceptability. In Eurocode-8, the deformation capacity of beam-columns is expressed in terms of chord rotations and three limit states are considered. In FEMA-356, plastic rotation angles with respect to three different performance levels are of concern. According to TEC-2007, the ultimate strain values in the extreme fibers of concrete and steel are specified for the evaluation criteria and three performance levels are defined.

It should be stated that, beam moment capacities exceed those of columns in the SPEAR building; consequently all beams are observed to be in the elastic range according to both response history analysis and test results. Therefore, adequacy of the acceptance criteria of the aforementioned procedures can be checked only for column members.

The second story is the critical level according to test results that are also supported by the analytical results. Therefore, results are presented for the second story in the following sections.

4.1 Evaluation according to Eurocode-8 Acceptance Criteria

The damage levels of flexural critical members are assessed according to the chord rotation values that the member experiences under the given ground motion. The chord rotations from the analysis are compared with the capacities defined for each limit state (Table 3). For damage limit state (DL), the chord rotation is given by the chord rotation at yielding, θ_y, computed from Eqn. 4.1:

$$\theta_y = \varphi_y \frac{L_v}{3} + \alpha_{el} + \alpha_{sl} \frac{0.2\varepsilon_{sy} d_b f_y}{(d - d')\sqrt{fc}} \tag{4.1}$$

where the first two terms account for flexural and shear contributions, respectively, and the third accounts for the anchorage slip of bars. d and d' are the depth to the tension and compression reinforcement, respectively, and f_y and f_c are the estimated values of the tensile strength of reinforcement and the concrete compressive strength, respectively. α_{el} is taken as 0.00275 for beams and columns. α_{sl} is a variable that is associated with the slip condition of the longitudinal l reinforcement. If slip occurs in the longitudinal reinforcement, α_{sl} is taken as 1, otherwise it is assumed to be equal to 0 [9].

In the test building, the observations and the use of smooth bars as the longitudinal reinforcement imply that bond-slip caused the rotation at the column ends and contributed to the story drift. However, as the bond-slip behavior is not simulated in the analytical model, α_{sl} is assumed to be 0 in the interest of consistency.

The chord rotation related to the severe damage (SD) is assumed as the 75 % of the ultimate chord rotation θ_{um} given in Eqn. 4.2:

$$\theta_{um} = \frac{1}{\gamma_{el}} 0.0172 (0.3^{\upsilon}) \left[\frac{\max(0.01, \omega')}{\max(0.01, \omega)} f_c \right]^{0.175} \left(\frac{L_v}{h} \right)^{0.4} 25^{\left(\alpha \rho_{sx} \frac{f_{yw}}{f_c}\right)} \left(1.3^{100\rho_d}\right) \tag{4.2}$$

Here γ_{el} is equal to 1.5 for primary elements, h is the depth of cross-section, $\nu = N/bhf_c$ (b is the width of the compression zone, N is the axial force (positive for compression), f_c is the estimated value of the concrete compressive strength (MPa), ω and ω' are mechanical reinforcement ratio of the tension and compression longitudinal reinforcement respectively, L_v is the shear span (M / V = moment /shear), $\rho_{sx} = A_{sx} / b_w s_h$ is the ratio of transverse steel parallel to the direction of loading (s_h = stirrup spacing), f_{yw} is the estimated yield strength of transverse reinforcement, ρ_d is steel ratio of diagonal reinforcement (if any), in each diagonal direction and α is the confinement effectiveness factor.

As the confinement and diagonal reinforcement are almost non-existent in the specified members, α and ρ_d was taken as zero.

The calculated chord rotation capacities at damage limit state (DL) and severe damage limit state (SD) are compared with the demand values of analytical results as shown in Table 3.

Table 3 Performance levels of second story columns according to Eurocode- 8

Member	$\theta_{capacity}$ DL (rad)	$\theta_{capacity}$ SD (rad)	θ_{demand} (rad)	Performance Level
C1	0.0091	0.0257	0.0111	DL - SD
C2	0.0090	0.0260	0.0111	DL - SD
C3	0.0098	0.0235	0.0112	DL - SD
C4	0.0093	0.0250	0.0112	DL - SD
C5	0.0085	0.0277	0.0116	DL - SD
C6	0.0044	0.0179	0.0108	DL - SD
C7	0.0086	0.0273	0.0125	DL - SD
C8	0.0084	0.0279	0.0127	DL - SD
C9	0.0089	0.0264	0.0116	DL - SD

DL : Damage limit state, SD : Severe damage limit state

4.2 Evaluation according to FEMA-356 Acceptance Criteria

The maximum plastic rotation values are obtained from the bi-directional nonlinear time history (0.15g PGA) analysis results. Beams and columns are both accepted as primary components. Both experimental and analytical results indicate that the beams would remain within the elastic range. Hence no plastic rotation demands are calculated for beam elements. As the stirrup configuration is inadequate and confinement effect is almost non-existent, plastic rotation limits with respect to the non-conforming transverse reinforcement are used.

The results in the most critical direction are shown in Table 4. Plastic rotation limits corresponding to different performance levels are calculated by making linear interpolation between specified values considering the axial load and shear force in each direction. We note that the measured demands hand down a harsh verdict for performance in this procedure.

Table 4 Performance levels of second story columns according to FEMA-356

Member	$\theta_{capacity}$ IO (rad)	$\theta_{capacity}$ LS (rad)	$\theta_{capacity}$ CP (rad)	θ_{demand} (rad)	Performance Level
C1	0.005	0.005	0.005	0.0061	> CP
C2	0.005	0.005	0.005	0.0051	> CP
C3	0.0043	0.0043	0.0053	0.0081	> CP
C4	0.0048	0.0048	0.0058	0.0059	> CP
C5	0.005	0.005	0.005	0.0052	> CP
C6	0.005	0.005	0.005	0.0101	> CP
C7	0.005	0.005	0.005	0.0056	> CP
C8	0.005	0.005	0.005	0.0051	> CP
C9	0.005	0.005	0.005	0.0065	> CP

IO: Immediate occupancy, LS : Life safety, CP : Collapse prevention

4.3 Evaluation according to Turkish Earthquake Code Acceptance Criteria

The damage levels are defined by specifying three limit states: minimum damage limit state (MDL), safety limit state (SL) and collapse limit state (CL) respectively. Each limit state is expressed in terms of strain values in the concrete extreme fibers and reinforcement steel of the reinforced concrete members.

The acceptance limit values are calculated by considering the amount of transverse reinforcement in the examined sections. In the test building, the stirrups are closed with 90° angle hooks instead of 135° angle of those specified in the code. In addition to that, the stirrup configuration is inadequate and no confinement zone exists in the beam-column joints. As a result of these statements, strain values corresponding to the limit states are defined in equation (3).

$$\begin{aligned} (\varepsilon_{cu})_{MDL} &= 0.0035 \quad ; \quad (\varepsilon_s)_{MDL} = 0.010 \\ (\varepsilon_{cu})_{SL} &= 0.0035 \quad ; \quad (\varepsilon_s)_{SL} = 0.040 \\ (\varepsilon_{cu})_{CL} &= 0.0040 \quad ; \quad (\varepsilon_s)_{CL} = 0.060 \end{aligned} \tag{3}$$

According to the results of analysis, concrete fiber strain values are below the specified limit values, thus, the strain values in the steel fibers are considered. The capacity and demand values are shown in Table 5. We note that in many cases material strains must be back-calculated by the user. This is a potential source of error.

Table 5 Performance levels of second story columns according to TEC-2007

Member	$(\varepsilon_s)_{capacity}$ **MDL**	**SL**	**CL**	$(\varepsilon_s)_{demand}$	**Performance Level**
C1	0.01	0.04	0.06	0.0134	MDL - SL
C2	0.01	0.04	0.06	0.0124	MDL - SL
C3	0.01	0.04	0.06	0.0172	MDL - SL
C4	0.01	0.04	0.06	0.0122	MDL - SL
C5	0.01	0.04	0.06	0.0139	MDL - SL
C6	0.01	0.04	0.06	0.0218	MDL - SL
C7	0.01	0.04	0.06	0.0157	MDL - SL
C8	0.01	0.04	0.06	0.0140	MDL - SL
C9	0.01	0.04	0.06	0.0169	MDL - SL

MDL: Minimum damage limit, SL: Safety limit, CL: Collapse limit

5. COMPARISON BETWEEN ANALYTICAL AND EXPERIMENTAL RESULTS

Damage pattern identified through visual inspections after the test (0.15 PGA) showed that only light damage occurred at top ends of 2nd story columns. The analytical response results are consistent with those of the experiment. However, there are differences between the performance level qualifications according to the document to which reference is made.

The chord rotation demands calculated according to the element drift ratios specified in Eurocode-8 are slightly larger than the chord rotation capacities given at damage limit (DL) as shown in Table 3. It can be concluded that the column members are just at the beginning of the inelastic response. This is consistent with the experimental observations.

According to FEMA-356 provisions, plastic rotation demands exceed the collapse prevention limits; therefore all second story columns are predicted to be in the collapse region. This is an unexpected outcome because the specified plastic rotation capacity limits are considered, it is inferred that these values are much smaller in the case of nonconforming transverse reinforcement. It is concluded that FEMA-356 results are very conservative for the condition of nonconforming transverse reinforcement.

The outcomes of evaluation considering the ultimate strain values in the extreme fibers of concrete and steel (TEC-2007) are between those of Eurocode-8 and FEMA-356. The limit strain value (ε_{cu} = 0.0035) in the extreme fibers of concrete is the same for minimum damage (MDL) and safety (SL) levels considering the inadequate transverse reinforcement configuration. However, the demand values are lower than these limits. Hence, the controlling criterion for comparison is estimated to be the strain in the extreme fibers of steel. The limit values for the minimum damage level (MDL) and safety level (SL) are 0.01 and 0.04 respectively. The demand values are larger than those of MDL. The results of TEC-2007 are similar to those of Eurocode-8 but demand-to-capacity ratios of TEC-2007 are predicted to be larger than those of Eurocode-8.

To compare the capacity and demand values of TEC-2007 and FEMA-356 in a more rational way, the corresponding strain values are calculated (Table 6) according to the plastic rotation values given in Table 4. Even in the collapse prevention level, the corresponding steel strain is 0.01 which is equal to the steel strain value for the minimum damage limit state of TEC-2007. For this reason, any member estimated to be in the MDL – SL range (TEC-2007) is considered to be beyond the collapse prevention level according to the FEMA-356 procedure. It can be concluded that FEMA-356 results are more conservative than those of TEC-2007 for nonconforming transverse reinforcement condition.

Table 6 Corresponding strain values in the extreme fibers of concrete and steel

Member	IO		LS		CP		Demand	
	ε_{cu}	ε_s	ε_{cu}	ε_s	ε_{cu}	ε_s	ε_{cu}	ε_s
C1	0.0023	0.0107	0.0023	0.0107	0.0023	0.0107	0.0025	0.0113
C2	0.0021	0.0097	0.0021	0.0097	0.0021	0.0097	0.0022	0.0103
C3	0.0029	0.0082	0.0029	0.0082	0.0032	0.0096	0.0040	0.0132
C4	0.0024	0.0093	0.0024	0.0093	0.0027	0.0107	0.0026	0.0104
C5	0.0016	0.0100	0.0016	0.0100	0.0016	0.0100	0.0016	0.0103
C6	0.0012	0.0100	0.0012	0.0100	0.0012	0.0100	0.0017	0.0182
C7	0.0017	0.0099	0.0017	0.0099	0.0017	0.0099	0.0018	0.0108
C8	0.0015	0.0101	0.0015	0.0101	0.0015	0.0101	0.0015	0.0103
C9	0.0020	0.0098	0.0020	0.0098	0.0020	0.0098	0.0023	0.0121

6. CONCLUSIONS

In this paper we have reported the calculated performance of an asymmetric multi-story test building according to nonlinear procedures described in Eurocode-8, FEMA-356 and TEC-2007. Comparison of calculated results with the test measurements is summarized in Table 7. The demand values were obtained through bi–directional nonlinear time history analysis (0.15 PGA). Assessment was done at member level by considering the restrictions stated for inadequate transverse reinforcement configuration. The acceptance criteria and demand definition of Eurocode-8 are in good agreement with the experimental results. The performance levels determined by the TEC-2007 are almost the same with those of Eurocode-8, whereas the DCRs calculated by TEC-2007 are larger than those for Eurocode-8. The FEMA-356 procedure seemed to result in very conservative values so that all columns were found to be beyond the collapse prevention level. The reason of this outcome is that, the collapse prevention limit of FEMA-356 corresponds to the minimum damage limit of TEC-2007 for the considered building. It should be noted that strain limits specified in TEC-2007 are likely to be extremely variable in actual circumstances, so reliance on aggregated strains expressed as, e.g., rotations should be preferred.

This study indicates that the main difference between the procedures arises from differences of definition in the limit state values even in the case of a specimen with well known properties tested under tightly controlled circumstances. The variability under field conditions is likely to be much higher because properties of existing buildings and the precise ground motions to which they have been subjected are typically known only approximately. It is recommended that further experimental studies as well as the analytical studies be employed to understand the behavior of buildings under seismic actions. This way, structural performance can be calibrated better and accurate predictions can be made of damage in structural members.

Table 7 Comparison of assessment procedures with the experimental results

Member	Performance Levels			Experiment
	Eurocode 8	TEC 2007	FEMA 356	
C1	DL - SD	MDL - SL	> CP	Light Damage
C2	DL - SD	MDL - SL	> CP	Light Damage
C3	DL - SD	MDL - SL	> CP	Light Damage
C4	DL - SD	MDL - SL	> CP	Light Damage
C5	DL - SD	MDL - SL	> CP	Light Damage
C6	DL - SD	MDL - SL	> CP	Light Damage
C7	DL - SD	MDL - SL	> CP	Light Damage
C8	DL - SD	MDL - SL	> CP	Light Damage
C9	DL - SD	MDL - SL	> CP	Light Damage

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