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DO THE EUROCODE 8 FORCE-BASED DESIGN PROVISIONS LEAD TO THE SAFE AND PREDICTABLE SEISMIC RESPONSE OF RC FRAME BUILDINGS?

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ABSTRACT : Performance-Based Design is now widely recognized as the pre-eminent seismic design methodology for structures. The question thus naturally arises: Are Force-Based Design provisions of modern seismic codes compatible with Performance-Based Design objectives? This paper is an investigation into the predictability of response and margin of safety of designs provided by the Eurocode 8 seismic design provisions. Assessment is made by comparing the design displacements and forces for regular RC moment frame structures to those obtained from non-linear time-history analysis. Time-history analyses were performed using suites of real and spectrum compatible artificial accelerograms. It is noted that a single-valued force reduction factor is incompatible with the force demands exhibited in buildings incorporating the same seismic resisting system, only differing in their fundamental period. Furthermore, drift ratios and member curvature demands indicate that seismic performance levels are non-uniform for the same class and type of structure. These findings suggest a review of the Force-Based Design provisions of seismic codes to bring about improved predictions of displacements and forces, and thus expected structural response and performance.

KEYWORDS: Seismic response, RC moment frames, Performance-Based Design, time-history analysis, Eurocode 8

1. INTRODUCTION

The advent of Performance-Based Design methodologies now require that engineers develop code compliant structures that also achieve specific performance objectives. Accordingly, it is necessary to develop efficient designs with predictable seismic response. To this day, the seismic designs of most general and some complex building structures are performed with Force-Based Design (FBD) method. This method is conceptually straightforward and thus appealing, but relies heavily upon unique force-reduction factors and displacement equivalences for a selected lateral force resisting structural system. The FBD methodology may yield life-safe designs in most cases; however, its ability to deliver designs that achieve specific performance objectives remains in question. These issues of life-safety and predictable response are addressed in this paper through an investigation of a modern-day FBD code.
Three reinforced concrete frame buildings (four, eight, and 16-storey) were designed using the Eurocode 8 (EC8) FBD provisions (CEN, 2003). The building designs and detailing correspond to EC8 ductility class DCM (medium ductility). The seismic setting, in terms of the EC8, consisted of the very stiff soil type A, the large magnitude type I spectrum, and an earthquake design intensity of 0.35g PGA. The linear modal response analysis was employed to determine the code predicted story drift ratios and beam moment demands. Capacity design per the EC8 was implemented to design the beam shear reinforcement and the columns. The results of the code-based response spectrum analyses were subsequently compared to time-history analysis (THA) results of the same frames subjected to a suite of seven real and seven spectrum-compatible, artificial accelerograms. A total of four response parameters were investigated and reported on: (1) interstorey drift ratios, (2) curvature ductility demands, (3) storey shears, and (4) storey overturning moments.

2. THE STUDY

2.1 Case-study Buildings

The plan geometry and a generalized elevation of the case-study buildings are presented in Figure 1. The buildings consist of three, 7.5m long bays in each direction and have 3.5m storey heights. The lateral force resisting system (LFRS) consists of perimeter RC moment-resisting frames (MF). The floor masses for each building are given in Table 1. The masses were lumped at each floor level and included allowances for the non-structural elements such as partitions and ceilings. Since the buildings are symmetrical about the principal axes of the structure, the design was only carried out for one frame elevation (i.e. 2-D analysis). However, bi-axial flexural strength reductions were taken on the corner MF columns in accordance with EC8 section 5.4.3.2.1(2).

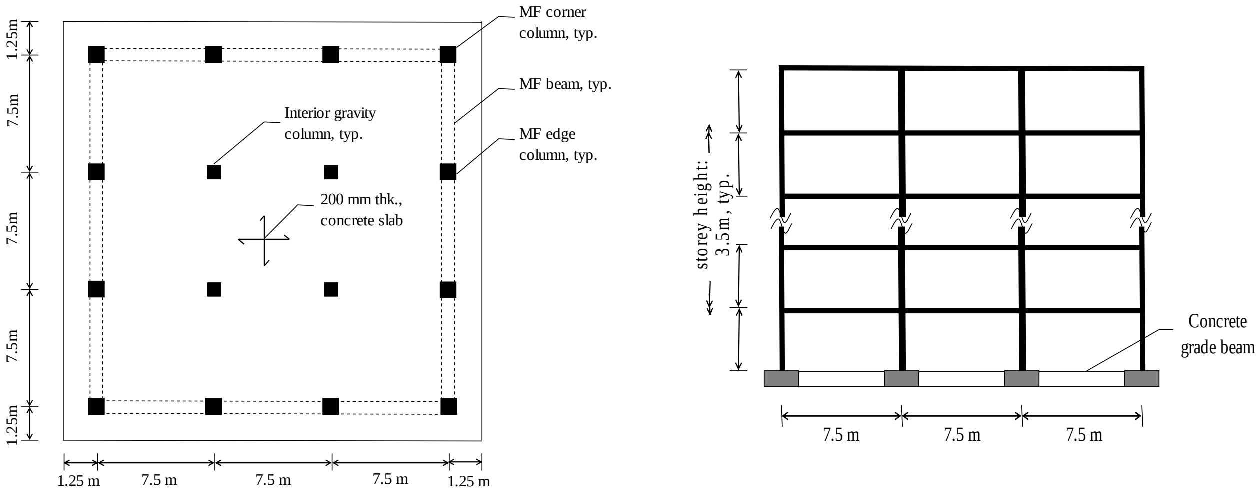

Figure 1 Typical framing plan (left) and elevation (right) of the case-study RC moment frame buildings

Design and mean material property values were determined from the code prescribed partial factors and probability distribution functions. The concrete and steel characteristic strengths were 35 MPa and 440 MPa, respectively. Design values were used for the code based design, along with the EC8 specified load combinations. The mean material values were used in the development of the THA analytical models.

Table 1.1 Case-study building floor masses

	4-Storey [kPa]	8-Storey [kPa]	16-Storey [kPa]
Floor Mass	7.40	7.71	8.60 / 7.82
Note: The two tabulated values for the 16-storey building are for levels 1 to 8 and 1 to 16, respectively.			

2.2 Eurocode 8 Force-Based designs

The fundamental concept behind FBD is that both the linear-elastic and non-linear response of structures incorporating the same lateral force resisting system can be correlated through a force-reduction factor (termed "behavior" factor, *q* in EC8). The concept is depicted in Figure 2. The EC8 prescribes a *q* of 3.9 for this system. In general, the design acceleration spectrum is derived by dividing the elastic spectrum ordinates by the force-reduction factor (ref. Figure 2). As shown in Figure 3 (left), the EC8 design spectrum includes some modifications to this approach- very short period ordinates are increased and a minimum spectral acceleration is prescribed (equal to 0.2PGA = 7.0%*g*). The consequence, for the case under study, is that buildings with fundamental periods greater than 1.28s will have the same first mode base shear contribution.

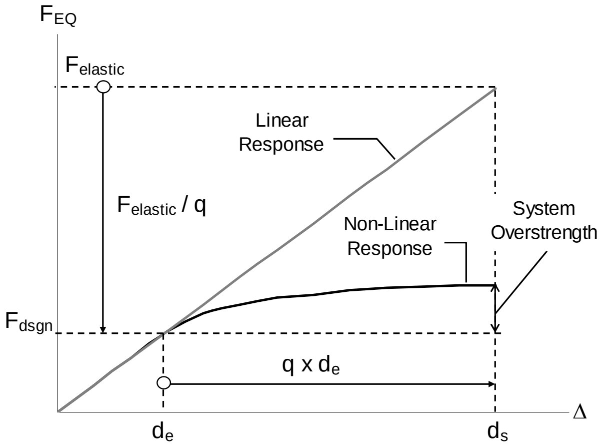

Figure 2 Force-Based Design concept

For period T < 4.0sec, the elastic acceleration spectrum can be converted to the displacement spectrum from the following expression:

$$S_D(T) = S_a(T) \cdot \left(\frac{T}{2\pi}\right)^2 \tag{2.1}$$

When a linear analysis is performed with the design forces, the displacement of a point in the structural system (d_s) during the design basis earthquake is estimated by multiplying the analysis value d_e by the displacement behavior factor q_d (equal to q for medium-to-long period structures per EC8). Moreover, EC8 states that the value of d_s does not need to be larger than the value derived from the elastic spectrum $d(S_D)$.

$$d_s = q_d \cdot d_e \le d(S_D) \tag{2.2}$$

By substituting Eqn. (3.1) into Eqn. (3.2), the "design" displacement spectrum may be defined as:

$$S_{Dd}(T) = q_d \cdot S_{ad}(T) \cdot \left(\frac{T}{2\pi}\right)^2 \le S_D(T) \tag{2.3}$$

Figure 3(right) plots the displacement spectrum with and without the limiting value of $S_D(T)$**,** and the elastic displacement spectrum for the case under study. It is observed that the minimum spectral acceleration ordinate of the design acceleration spectrum mentioned above would produce increasingly larger spectral displacements in comparison to the elastic spectrum for buildings with periods greater than 1.28s. This is not physically possible and it underlines the fundamental role of the condition $d_s \le d(S_{De})$. Unfortunately, we noticed that in some codes (e.g. the Italian Building Code (DM, 2008) and the Uniform Building Code (ICBO, 1997)) this limitation is omitted. On a final note, EC8 section 5.2.3.4 requires that structures with fundamental period greater than the corner period, T_C, possess a minimum curvature ductility capacity equal to $2q$-*1.*, i.e. 6.8 for the case under study. This results in detailing for a maximum curvature ductility demand of *6.8*. It is interesting to note that this curvature ductility capacity can usually be achieved with the low transverse steel reinforcement ratios often found in gravity dominated designs.

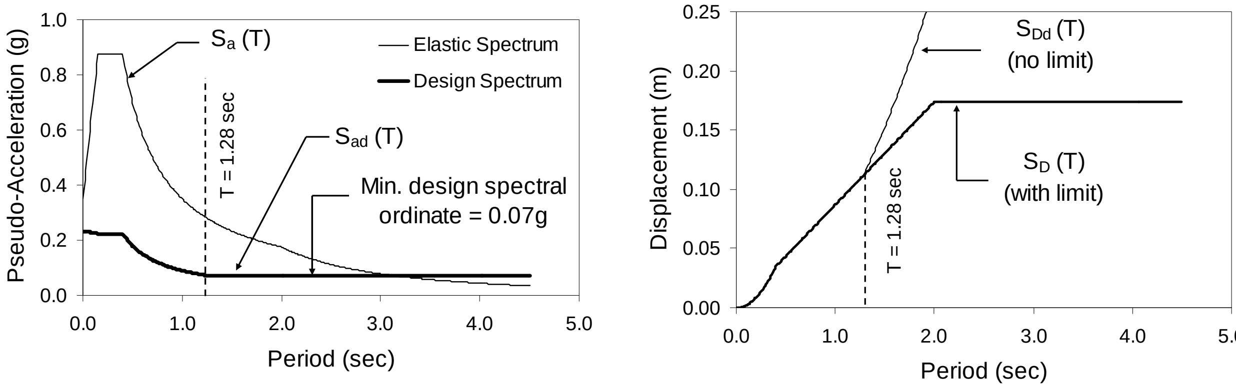

Figure 3. EC 8 elastic and design spectra (left) and interpretation of design displacement spectra (right)

2.3 Analytical models

In order to perform time-history analysis, models of the "as-designed" buildings were required. The structural analysis computer program RUAUMOKO (Carr, 2005) was used to determine structural response of the buildings. The main features of the analytical models included the use of mean material property values, confined concrete response, steel reinforcement strain hardening, and hysteretic behavior in the form of lumped plasticity at the ends of the frame beam and column elements. Flexural response of the beam and column plastic hinges was approximated with bi-linearized moment-curvature response. The Modified Takeda hysteresis rule (Otani, 1981) was used to represent the cyclically degrading response of the plastic-hinge zone. Rayleigh tangent stiffness proportional damping of 5% in modes 1 and 3 for the four-storey building, and 1 and 4 for the eight and 16-storey buildings was used. A dummy column was used to account for P-delta effects from the tributary gravity loads to the frame.

2.3 Accelerograms

Real and artificial accelerograms were used to conduct time-history analysis. The artificial accelerograms were generated using SIMQKE (Gasparini and Vanmarcke, 1976) and made compatible to the EC8 defined spectrum. The real accelerograms were selected from the Pacific Earthquake Engineering Research Center (PEER, 2005) strong motion database using a computer code developed by Dall'Ara *et al.* (2006). The acceleration spectra for the real and SIMQKE records are shown in Figure 4 (right). A blow up of the average acceleration spectra for each record type along with the fit to the EC8 elastic spectrum is shown in Figure 4 (left).

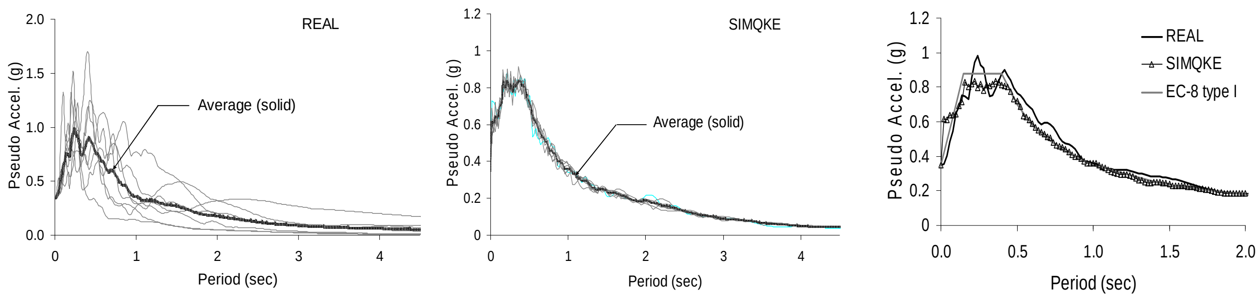

Figure 4 Real (left) and SIMQKE (center) acceleration spectra and comparison of the average acceleration spectra to the EC8 type I spectrum (right)

3. FINDINGS

3.1 Interstory drift ratios

The maximum linear and non-linear interstorey drift ratios were recorded for all three buildings. The results for the non-linear buildings subjected to the real (REAL in the figures) and artificial accelerograms (SIMQKE in the figures) are shown in Figure 5 along with the EC8 elastic response spectrum results (ELASTIC in the figures), in accordance with equation (2.2). The drift profile based on the elastic analysis displacements multiplied by the behavior factor (DESIGN x q in the figures) is also shown.
In the case of the four-storey building, the peak interstorey drift ratio (peak is here defined as the maximum value of all the storeys) is well predicted by both the design and elastic spectrum (since the fundamental period was near the threshold value of 1.28s). Moreover, they provide a satisfactory upper bound envelope for the THA results at all levels. The elastic spectrum provides good estimates of drifts with respect to THA for the eight and 16 storey buildings, showing an average error of +3% and -23%, respectively (plus indicates estimation is greater than the average of REAL and SIMQKE peak result). In contrast, if the design spectrum is used, the peak THA results are overestimated by +69% for the eight-storey building and +116% for the 16-storey building – confirming the necessity of the limitation in equation (2.2). In the case of the EC8 provisions, the engineer would revert back to the elastic spectrum results. However, as was previously mentioned, other building codes do not provide such guidance or measures against this error in calculation. The effect of such an oversight would be to redesign stiffer, stronger, and more costly structures that may well have met the code drift limit.

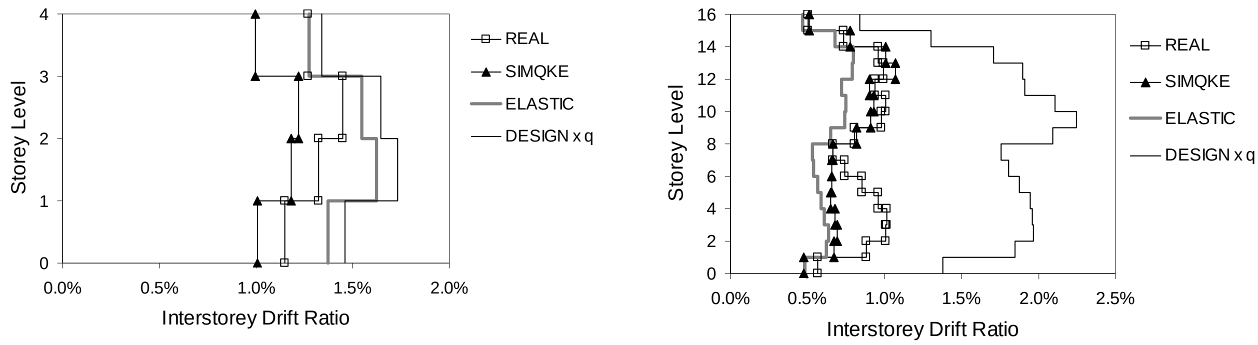

Figure 5 Comparison of the maximum interstorey drift ratios of the four (left) and 16-storey (right) building for the non-linear models

3.2 Curvature ductility demand

The maximum member end curvature ductility demands and distributions were found at all beam-column joints. In all but two cases, curvature ductility demands were shown to be less than 50% of the EC8 required value of 6.8. The peak curvature ductility demand and number of plastic hinge locations reduced with increase in fundamental period. Since the amount of confinement reinforcement required to meet the curvature ductility value of 6.8 is relatively small (and transverse reinforcement was often governed by capacity design shear demands), the EC8 value serves as good lower bound. Concerning the distribution of plastic hinges throughout the structures, a significant finding from these results (Figure 6) was that for almost all THA runs the EC8 strong column-weak beam criteria satisfactorily precluded the formation of column plastic hinges at the beam-column joints (base of columns not included). The curvature ductility distributions pointed to the fact that, in many instances, the base of the columns of the eight and 16-storey buildings did not form plastic hinges. In total, the base columns hinged in one out of the 14 THA runs for each building (compared to nine out of 14 for the four-storey). Therefore, the full the base hinge. This is attributed to the column sizes arrived at from the strong column-weak beam criteria, the use of design material strengths (rather than mean values), and typical detailing which extends the same vertical reinforcement from the foundation level through the first floor level, instead of using less at the base.

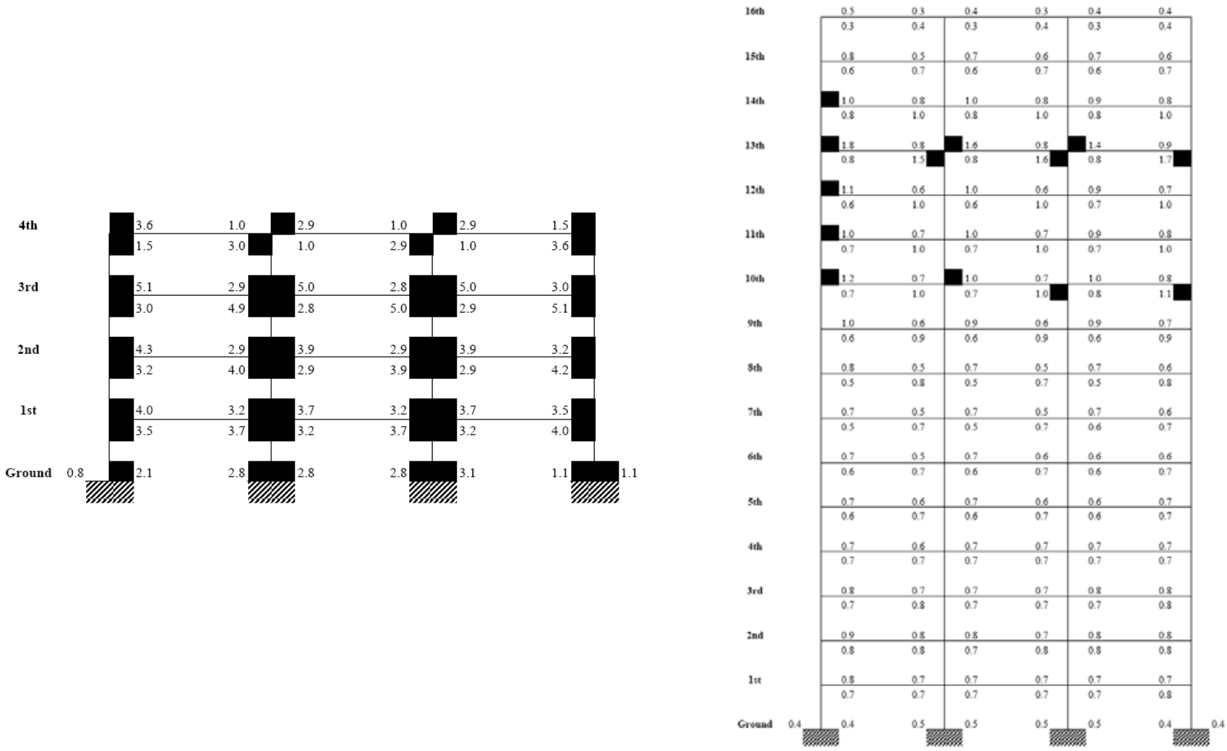

Figure 6 Maximum curvature ductility demands in four (left) and 16-storey (right) buildings for real accelerogram record number 1

3.3 Storey shears

The maximum storey shears were investigated for all three buildings. It is clear from Figure 7 that the EC8 response spectrum storey shears are significantly less than those resulting from THA. Overall, the clear trend is for the storey shear demand from THA to increasingly exceed the code design value with decrease in fundamental period (number of storeys). On average, the base shear demand exceeded the EC8 design base shear by 2.6, 1.8, and 1.3 times for the four, eight and 16-storey building, respectively. The large overstrength shear demands on the columns were not found to pose a shear failure threat due to the capacity design implemented. By calculation, the four, eight and 16-storey building base columns possess normalized base shear capacities of 43%W, 22%W, and 25%W, which are all well over the base shear demands shown in Figure 7. Nevertheless, the modal response spectrum analysis method proved futile in estimating the storey shears.
It is of interest to examine the difference between conducting response spectrum analysis with the aforementioned minimum spectral acceleration ordinate imposed by the EC8 and simply using the elastic spectrum divided by the behavior factor (ELASTIC/q in Figure 7). Solely using the reduced elastic acceleration spectrum yields very similar overstrength base shear demands (approximately +300%) in all three buildings when compared to THA results. This would suggest that *q* should be reduced considerably to better estimate force demands. Furthermore, it may be prudent to remove the minimum spectral ordinate provision from the code in future calibrations of the behavior factor to eliminate such inconsistencies in predicted force levels for the same class and type of structure. This would nullify the apparent period dependent trend noted above

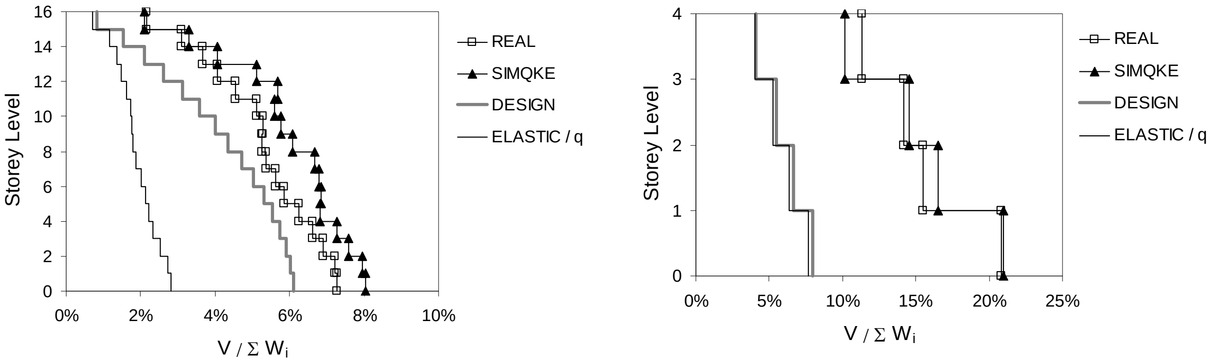

Figure 7 Comparison of the maximum storey shears for the four (left) and 16-storey (right) buildings

3.4 Storey overturning moments

Examination of Figure 8 shows the same trend as for the storey shears results – the overstrength demands from THA decrease (in comparison to the EC8 design values) with increase in the buildings' fundamental period, although to a lesser extent. On average, the ratios of THA result to the EC8 design value of base overturning moment were 2.3, 1.5, and 0.9 for the four, eight and 16-storey building, respectively. It is seen that for the 16-storey building, the response spectrum analysis design values are well in line with the THA results. The ASCE/SEI 7-05 (ASCE, 2006) design standard states that the force-reduction factor for a building can be computed from the ratio of the base OTM's from the linear and non-linear THA of the same structure to a given ground motion. Applying this definition to the studied cases, the average ratio of linear to non-linear base OTM values were 1.47 and 1.19 for the 4 and 16 storey buildings, respectively. These are significantly smaller than the code prescribed behavior factor of 3.9. This finding serves to reaffirm the suggested use of a lower behavior factor in design to better estimate the storey shears and overturning moments observed.

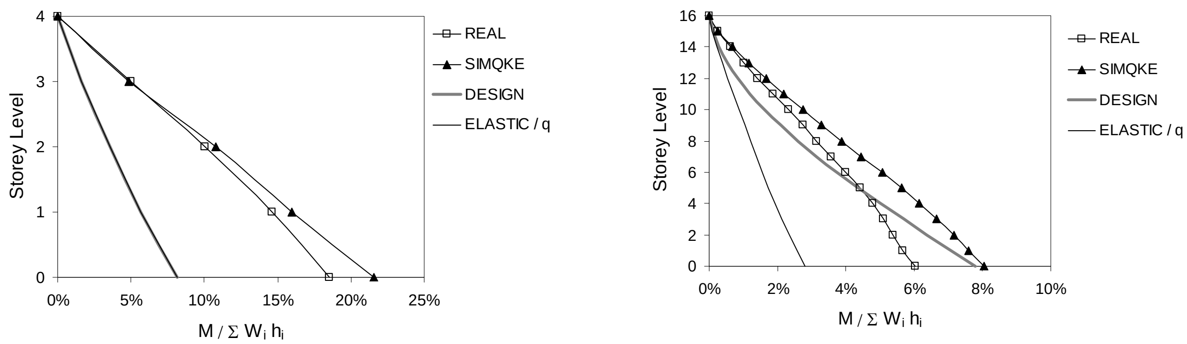

Figure 8 Comparison of the maximum storey overturning moments for the four and 16-storey buildings

4 RESULTS AND CONCLUSIONS

Time-history results indicated that the design of the moment-resisting concrete frames per the Eurocode 8 linear analysis provisions yield life-safe structures; yet prediction of displacements and forces. A correct evaluation of the members initial effective flexural stiffness (moment-curvature analysis or assumption of cracked section properties) and the upper bound limitation imposed in the calculation of the design displacements (d_s need not be larger than the value derived from the elastic spectrum) is of primary importance when estimating interstorey drifts, and hence predicted damage levels. Furthermore, the "one size fits all" approach in the selection of behavior factors may result in significant underestimations of storey shears and overturning moments, depending on the manner by which the design spectrum is defined. As a consequence, the degree of protection provided by the EC 8 under a given seismic intensity is non-uniform - even for the same ductility class and type of structure.

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