Paper No. 1820. (quote when citing this article)
Eleventh World Conference on Earthquake Engineering
ISBN: 0 08 042822 3

EVALUATION OF BEHAVIOUR FACTORS FOR RC BUILDINGS BY NONLINEAR DYNAMIC ANALYSIS

L. M. SALVITTI and A. S. ELNASHAI

Department of Civil Engineering, Imperial College of Science, Technology and Medicine, London, U.K.

ABSTRACT

Seismic design makes use of concepts of energy absorption and dissipation to reduce the design forces in order to achieve economy. The 'behaviour factor' (R in US practice and q in European practice) assumes a central role in the seismic design process, since design forces are the quotient of elastic forces and this all-important parameter. In this paper, the procedure for the evaluation of behaviour factors using nonlinear dynamic analysis techniques is outlined and an application to a set of Eurocode 8-designed reinforced concrete structures is presented. An alternative method for calculating behaviour factors using inelastic response periods is also presented and shown to give higher q-factors in comparison with the use of elastic periods and spectra.

KEYWORDS

Behaviour factors; limit state definitions; RC frame-wall system; inelastic response periods.

INTRODUCTION

In the context of modern earthquake-resistant design, the response modification (or behaviour) factor plays a fundamental role. This is used to derive the design acceleration response spectrum from its linear elastic equivalent, allowing the benefits offered by the energy dissipation capability of each structure to be availed of, while ensuring that the imposed ductility demand does not exceed the available supply. Such factors, referred to as R (or R_W for working stress) in US practice and q in European practice, account implicitly for inelastic response, the presence of damping and other force reducing effects, such as period elongation (or stiffness degradation) and soil-structure interaction.

The reliable estimation of the behaviour factor is therefore an essential ingredient in the seismic design recipe. The evaluation of this parameter involves subjecting the same structure to progressively increasing shaking intensity up to the attainment of (i) a yield limit state and (ii) an ultimate or collapse limit state. It follows that experimental evaluation of these important parameters is not possible and it is clear, therefore, that the only option left is nonlinear dynamic analysis. In this paper the procedures involved in such an endeavour are reviewed and application examples are given. The obtained values are compared to the design q-factors. Finally, an alternative procedure is outlined whereby inelastic response periods are used to evaluate behaviour factors.

STRUCTURAL SYSTEM AND DYNAMIC LOADING

The current European code of practice for the design and detailing of structures in seismic regions, Eurocode 8 (EC8), specifies maximum allowable q-factor values for a range of structural configurations and forms of construction. For the design of RC structures, three distinct classes of ductility ('L'ow, 'M'edium and 'H'igh) are defined with progressively higher values of behaviour factor and correspondingly more stringent detailing requirements, necessary to mobilise certain minimum ductility levels.

The analytical evaluation of behaviour factors is attempted in this paper by application to a coupled frame-wall structure. The same general layout is utilised in four distinct cases of ductility class and design peak ground acceleration. The 8-storey reinforced concrete structure considered has overall plan dimensions of 15m by 20m, a total height of 24m and equal storey heights of 3m. The buildings, which are designed and detailed to EC2 and EC8 (Fardis, 1994), are assumed to rest upon EC8-defined soil type B. The design q-values are obtained by the product of the basic behaviour factor (q_o) and the k_W (dominant wall failure mode), k_R (regularity) and k_D (ductility class) factors, equal to 1.0, 0.75 and 0.5 for Ductility Classes 'H', 'M' and 'L', respectively. The ensuing four design situations are summarised in Table 1 below.

Table 1. Seismic design situations considered

Model Suffix	Ductility Class	Design Peak Ground Acceleration (g)	Design Behaviour Factor
DCH030	H	0.30	3.50
DCM030	M	0.30	2.625
DCM015	M	0.15	2.625
DCL015	L	0.15	1.75

Actions due to gravity are combined with dynamic loading, by making use of four horizontal component EC8-generated acceleration records (herein referred to as EC8-1 through EC8-4). It is beyond the scope of this paper to discuss the relative merits of the use and validity of artificially-generated input motions. It suffices to state that these records have been used in order to allow effective comparison with the EC8 design and with other studies undertaken in Europe.

BEHAVIOUR FACTORS - DEFINITIONS

The behaviour factor is defined as the ratio of the ordinate of the elastic acceleration response spectrum expected at a site to that of an inelastic spectrum used for design of a structure with a particular fundamental elastic period, viz.

$$q_{code} = (S_a)_d^{el} / (S_a)_d^{in} \tag{1}$$

where $(S_a)_d$ is the spectral ordinate obtained from the response spectrum corresponding to the elastic fundamental period of the structure considered, as shown in Fig. 1 below. The superscripts 'el' and 'in' refer to elastic and inelastic values, respectively.

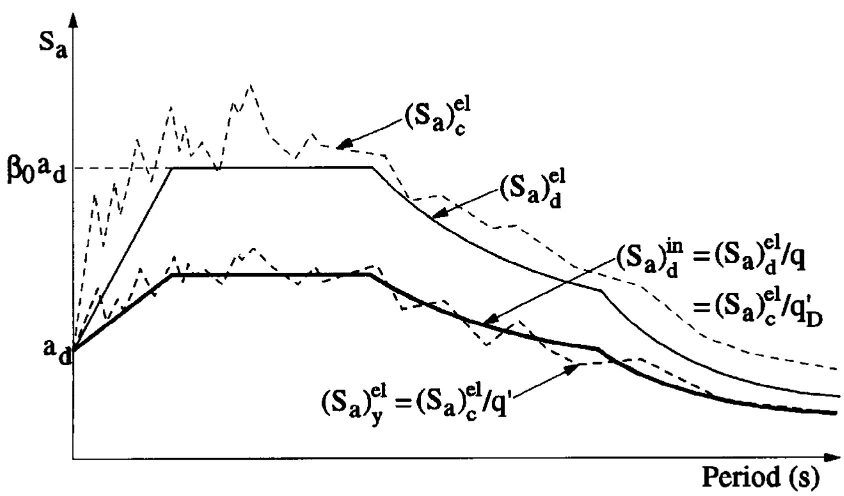

Fig. 1. Design, collapse and yield acceleration spectra

By assuming that collapse of the structure is imminent under the actions defined by the elastic design spectrum and that the point of yielding is equally imminent under the conditions defined by the inelastic design spectrum, then

$$q' = (S_a)_c^{el} / (S_a)_y^{el} \tag{2}$$

defines the behaviour factor by relating the elastic response spectra of the yield (subscript 'y') and collapse (subscript 'c') earthquakes. For a uniquely defined collapse state, combining eqs. (1) and (2) results in

$$q'_D = (S_a)_c^{el} / (S_a)_d^{in} \tag{3}$$

which has been utilised in previous studies.

Further, by assuming that the response spectra of the yield and collapse earthquakes display constant amplification, β_0 (ratio of response-to-peak ground acceleration = S_a/a_g), at least for the period considered, eqs. (4) and (5) result:

$$q' = a_{g\ (collapse)} / a_{g\ (yield)} \tag{4}$$

$$q'_D = [a_{g\ (collapse)} / a_{g\ (design)}] \cdot q_{code} \tag{5}$$

By comparing the above, it is noted that the definition given in eq. (5) checks the validity of the design by describing a general 'overstrength' term, accounting for the ability of the structure to resist greater loads due to provisions of additional capacity and ductility. This definition has the advantage of relating the intensity of loading at collapse to the loads used in design, but it has the shortcoming of not taking the disparity between ground motion at yield and collapse into account. The writers believe that eq. (4) gives the real analytical behaviour factor, whilst eq. (5) gives an indication of the extra seismic capacity. Current methods of evaluation of the behaviour factor make use of the above definitions and provide a reliable means of comparison with code-proposed values, justifying their use in the present paper. A detailed discussion of various other definitions relating specific limit states at collapse and at yield is given by Broderick (1994).

It is apparent that the concept of the behaviour factor has its shortcomings, such as ground motion dependence. Also, this parameter is considered to be period-independent, which will not be the case. Whereas the present paper employs the established techniques for the evaluation of behaviour factors using inelastic dynamic analysis, discussion of its validity and possible alternative approaches is constructive.

Firstly, ratios of yield and collapse earthquake spectra for different structural configurations and designs will not be constant, highlighting the structural dependence of the behaviour factors. Secondly, the dynamic amplification is not necessarily constant, even when a single structure subjected to a single earthquake is considered. Finally, the dominant structural response periods under the yield and collapse earthquakes are both far from their elastic counterparts and quite distinct as well as not readily predictable.

ANALYTICAL MODEL AND LIMIT STATE DEFINITION

The structures are modelled and analysed using the advanced computer code ADAPTIC (Izzuddin and Elnashai, 1989), which has been extensively tested and calibrated elsewhere (Madas and Elnashai, 1992; Elnashai and Izzuddin, 1993, amongst others). Elements accounting for the spread of plasticity across the section as well as the member length are combined with material models for concrete under active confinement and reinforcing steel with kinematic hardening (Elnashai and Izzuddin, 1993; Martinez-Rueda and Elnashai, 1995). The equations of motion are solved by implementation of the numerically dissipative Hilber-Hughes-Taylor α-integration scheme (Broderick *et al.*, 1994).

The regularity of the structures in plan and elevation allows a simplified two-dimensional analysis approach to be adopted, alleviating a rather onerous three-dimensional alternative. The main lateral force resisting system in the direction considered is a core, consisting of two channel-shaped shear walls, extending over the full height and coupled at each level by a pair of deep beams with a low shear span ratio. Moment frames provide lateral resistance at the perimeter locations. Combination of these two load-resisting systems is achieved by means of an 'overlay' approach, in which common finite element nodes are defined in one plane. Thus, frame and coupled wall members share the same rotational and translational degrees of freedom at the perimeter and wall external face locations, implying the assumption of infinite in-plane stiffness of the slab.

In order to assess the validity of the developed analytical models, eigenvalue analyses were carried out prior to the application of time-history loading. Fundamental elastic periods of vibration of 0.465, 0.464, 0.567 and 0.563 seconds were obtained for the DCH030, DCM030, DCM015 and DCL015 structures, respectively, by making use of the Lanczos algorithm. These showed good agreement with the values used in design, with a maximum variation of 6% (Fardis, 1994).

Static push-over analyses under a triangular distribution of lateral loads were performed for each structure in order to investigate the applicability of possible yield and collapse criteria. At the yield limit state, the general objective is to identify a state at which significant response modification occurs, from a near-elastic mode to the onset of inelastic deformations, whilst at the collapse condition, it is conservative to assume that failure of on member or one storey signifies failure of the structure as a whole. Following a review of limit state definitions (Salvitti and Elnashai, 1995), two yield and two collapse limit state definitions were selected. First yield of the main tensile reinforcement was observed for top steel at T-beam ends and top and bottom bars for all symmetrically-reinforced sections and mapped onto the obtained load-displacement curves. The chosen global yield definition was defined by assuming an equivalent elasto-plastic system with reduced stiffness, evaluated as a secant passing through 75% of the maximum load. At a member level, the critical concrete strain criterion defined by Dodd et *al.* (1994) was selected following a comparison with that suggested in EC8. Hence, the critical strain was defined as that corresponding to a 15% drop in the confined concrete stress on the descending branch of the stress-strain curve. The critical strains obtained by adopting the former method for the nine confinement levels considered, were, on average, 18% higher than those calculated with the latter definition. A maximum level of deformation at the global level was defined by a 3% interstorey drift limit.

EVALUATION OF BEHAVIOUR FACTORS

A large number of nonlinear dynamic analyses were performed in order to identify the peak ground accelerations causing yield and collapse of the four structures. All combinations of failure and yield were evaluated, enabling possible bounds on q-factors to be identified. Hence, the confined concrete critical strain (I) and 3% maximum interstorey drift (II) collapse criteria were in turn combined with the first (i) and global (ii) yield definitions. Behaviour factors q' and q'_D, evaluated by using eqs. (4) and (5), respectively, are presented in Tables 2 and 3 below.

Table 2. Behaviour factors for the 0.30g designs

	DCH030						DCM030					
Input Motion	q'(I/i)	q'(I/ii)	q'(II/i)	q'(II/ii)	q'_D(I)	q'_D(II)	q'(I/i)	q'(I/ii)	q'(II/i)	q'(II/ii)	q'_D(I)	q'_D(II)
EC8-1	9.75	4.28	13.80	6.05	8.89	12.72	5.76	3.00	7.31	3.80	6.56	8.31
EC8-2	10.00	2.88	11.87	3.42	8.75	10.38	9.55	2.69	10.00	2.81	7.52	7.87
EC8-3	7.60	2.61	12.50	4.28	8.52	14.00	4.73	2.36	6.47	3.23	6.21	8.48
EC8-4	8.70	2.74	9.88	3.11	8.63	9.80	7.00	2.08	7.82	2.32	6.74	7.52
Mean	9.00	3.20	12.00	4.20	8.70	11.72	6.76	2.53	7.90	3.04	6.76	8.04
Mean/Design	2.57	0.91	3.43	1.20	2.48	3.35	2.57	0.96	3.01	1.16	2.57	3.06

Table 3. Behaviour factors for the 0.15g designs

	DCM015						DCL015					
Input Motion	q'(I/i)	q'(I/ii)	q'(II/i)	q'(II/ii)	q'_D(I)	q'_D(II)	q'(I/i)	q'(I/ii)	q'(II/i)	q'(II/ii)	q'_D(I)	q'_D(II)
EC8-1	5.33	2.52	9.89	4.68	8.40	15.57	4.52	1.85	6.43	2.64	6.07	8.64
EC8-2	6.37	2.68	10.00	4.21	8.92	14.00	5.58	1.85	8.72	2.88	5.60	8.75
EC8-3	5.68	2.00	8.95	3.15	9.45	14.87	4.60	1.44	8.00	2.50	5.37	9.33
EC8-4	8.33	3.12	13.17	4.94	8.75	13.82	6.71	1.71	10.57	2.69	5.48	8.63
Mean	6.43	2.58	10.50	4.24	8.88	14.56	5.35	1.71	8.43	2.68	5.63	8.84
Mean/Design	2.45	0.98	4.00	1.61	3.38	5.54	3.06	0.98	4.82	1.53	3.22	5.05

With reference to the mean-to-design behaviour factors presented in Tables 2 and 3 above, it is noted that lower (most conservative) bounds on both q' and q'_D are obtained by employing the member failure and global yield criteria. The following observations are also worthy of consideration. Firstly, the first yield criterion should be viewed in isolation, since it only provides a benchmark in assessing the validity of the analytical model. It also does not herald a marked change in the load-carrying response of the structure considered. Hence, q' values obtained by the global yield criterion are better representations of the inherent behaviour factor, whilst those obtained by the first yield criterion are related to the design quantities.

The most significant values of behaviour factor are, in the writers' opinion, q'(I/i) and $q'_D(I)$, ranging between 2.45 and 3.38. From this viewpoint, it is clear that the behaviour factors used in design may be increased without adverse effects on structural performance. This observation is subject to uncertainties in the modelling, analysis and assessment, such as 3D effects, foundation modelling, inclusion of infill panels and simultaneous application of both horizontal and vertical earthquake motion. The inclusion of such effect may affect the values obtained and require further assessment.

Alternative Approach

The strict definition of the behaviour factor, given in eq. (1), immediately suggests an alternative procedure based on the spectra of the two earthquakes, and not peak ground acceleration. Furthermore, the effect of utilising the predominant response period, and not the elastic value, is worthy of consideration. An alternative procedure is outlined in Fig. 2 below.

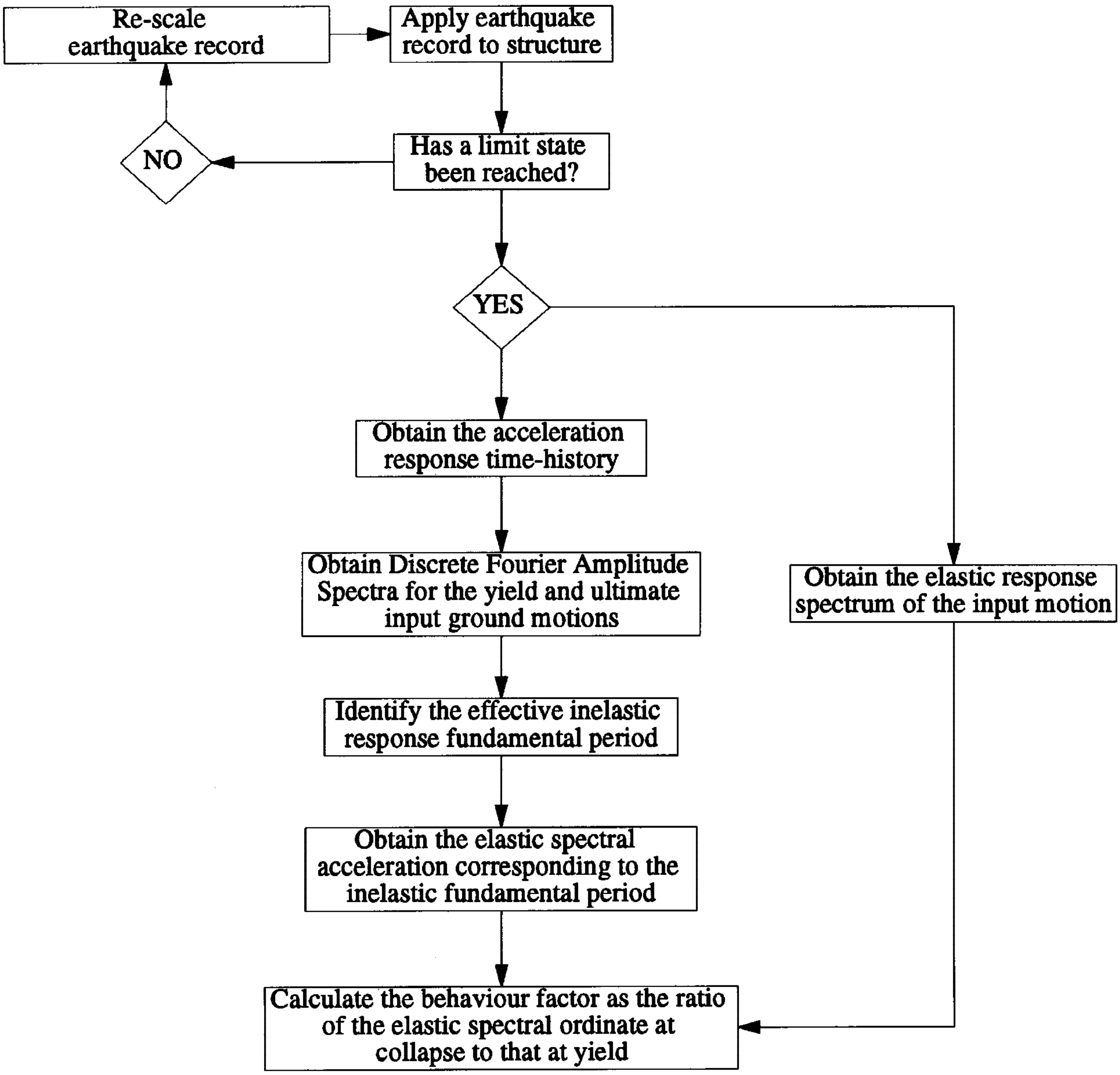

Fig. 2. Alternative procedure for the evaluation of the behaviour factor

As a result of stiffness degradation, the observed modal frequencies should be lower than their elastic counterparts. In Fig. 3 below, the response of one of the buildings at the design and twice the design intensity is shown, indicating period elongation with increasing input peak ground acceleration. Hence, the elongated fundamental periods at the yield (T_{1y}) and collapse (T_{1c} which is greater than T_{1y}) are entered in their respective elastic response spectra and corresponding spectral ordinates obtained ($(S_a)_{T_{1y}}$ and $(S_a)_{T_{1c}}$). The behaviour factor for the structure and input motion considered is then given by eq. (7):

$$q'_R = (S_a)^{el}_{T_{1c}} / (S_a)^{el}_{T_{1y}} \tag{7}$$

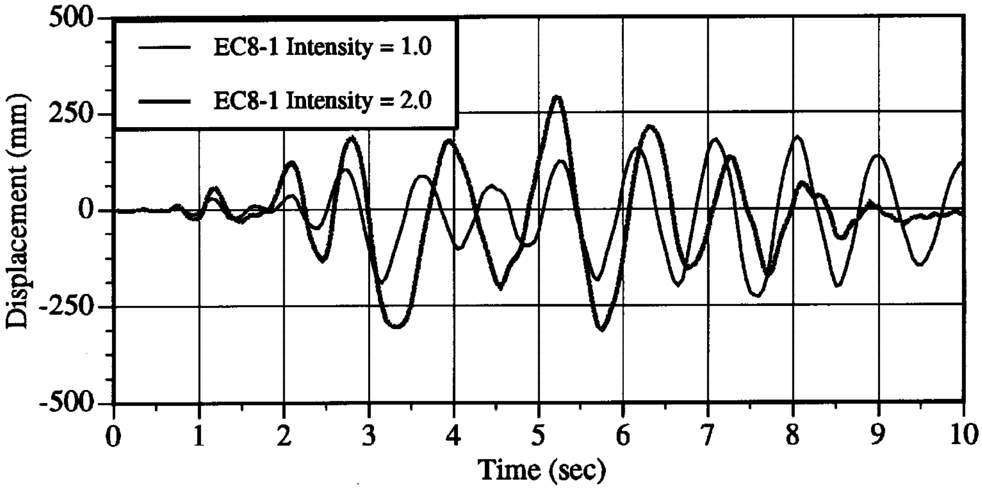

Fig. 3. Response period elongation with increasing ground shaking (from Salvitti and Elnashai, 1995)

This procedure was carried out for the four structures considered under the EC8-1 input motion. Discrete Fourier Amplitude Spectra for the global yield and collapse accelerations were obtained and nonlinear periods, corresponding amplitudes and spectral ordinates at the global yield ((ii)) and member failure ((I)) intensities are presented in Table 4 below.

Table 4. Alternative behaviour factor (q'_R) calculation results

	DCH030	DCM030	DCM015	DCL015
$a_{g\,(yield)}$ (g)	0.18	0.25	0.19	0.28
$a_{g\,(collapse)}$ (g)	0.77	0.75	0.48	0.52
T_{1yield} (s)	0.82	0.83	0.91	0.91
T_{2yield} (s)	0.24	0.28	0.26	0.27
$Sa(T_1)_{yield}$ (g)	0.335	0.463	0.337	0.497
$Sa(T_1)_{collapse}$ (g)	1.401	1.330	0.703	0.724
q'_R	4.18	2.88	2.08	1.46
q'_R/q'	1.02	1.04	1.21	1.27

Using the spectral ordinates presented in the table above, the obtained behaviour factors using eq. (7) are lower than the corresponding values based on elastic periods (Tables 2 and 3). Examination of the amplification factors sheds light on the variation in the margin of error in calculating q', compared to q'_R, indicated on the penultimate row of Table 4. The ratio of β_0 at yield to that at collapse follows precisely the ratio q'_R/q' (being 1.02, 1.04, 1.21 and 1.27 for the DCH030 through DCL015 structures). Further consideration of the results and their implications on the safe and economic evaluation of q-factors is accounting for contributions from higher modes and the effect of the ductility class.

CONCLUSIONS

Four RC structures were analysed under the effect of four artificial acceleration time-histories derived from the EC8 spectrum. The analyses comprised successive scaling of the records up to the satisfaction of a set of criteria enveloping the limit states of yield and collapse. The definition of behaviour factors based on peak ground acceleration at yield and collapse (q') gives values slightly below EC8-recommended q-factors. On the other hand, a definition which includes for the overstrength at the yield limit state (q'_D) results in values more than twice the EC8 recommendations. Collectively, the results indicate that behaviour factors in EC8 may be safely increased, in recognition of the satisfactory performance of the structures at excitation levels above twice the design event. Furthermore, since columns were never critical, it is reasonable to suggest that some reduction in column over-design factors will not have an adverse effect on seismic integrity of the buildings examined. These conclusions are clearly constrained by the limited study undertaken.

Conventional evaluation of response modification factors for a particular set of structures assumes that use of the elastic period is either accurate or conservative. Moreover, a constant amplification factor is assumed (ratio of ground-to-response acceleration) for both the yield and the collapse limit states. These two issues are briefly examined in the paper, by utilising the predominant inelastic response period (evaluated from Discrete Fourier Amplitude Spectra) in behaviour factor calculations. For the very limited sample studied above, it seems likely that q' values calculated by elastic periods and constant amplification are on the unconservative side, especially for structures designed to low values of ground acceleration. Further work is underway at Imperial College to explore the effect of higher mode contributions, design acceleration and ductility class on behaviour factors.

ACKNOWLEDGEMENT

The work was carried out as part of the collaborative research programme 'Prenormative Research in Support of Eurocode 8' funded by the European Commission, to which the writers are grateful. The work has benefited from discussions with Professors M.Fardis (University of Patras), P.E.Pinto (University of Rome), G.M.Calvi (University of Pavia) and Dr.E.Carvalho (LNEC).

REFERENCES

Broderick, B. M. (1994). Seismic Testing, Analysis and Design of Composite Frames. *PhD Thesis, Imperial College, University of London, UK.*

Broderick, B. M., A. S. Elnashai and B. A. Izzuddin (1994). Observations on the Effect of Numerical Dissipation on the Nonlinear Dynamic Response of Structural Systems. *Engineering Structures*, **16(1)**, 51-61.

Dodd, S. G., A. S. Elnashai, B. A. Izzuddin and G. M. Calvi (1994). A 3D Nonlinear Time History Analysis of a Curved Bridge. *Second International Workshop on Seismic Design of Bridges, Queenstown, New Zealand*, 617-639.

Elnashai, A. S. and B. A. Izzuddin (1993). Modelling of Material Nonlinearities in Steel Structures Subjected to Transient Dynamic Loading. *Earthquake Enginering and Structural Dynamics*, **22**, 509-532.

Fardis, M. N. (1994). Analysis and Design of Reinforced Concrete Buildings According to Eurocodes 2 and 8. *Prenormative Research in Support of Eurocode 8.*

Izzuddin, B. A. and A. S. Elnashai (1989). ADAPTIC, A Program for Adaptive Large Displacement Elastoplastic Dynamic Analysis of Steel and Composite Frames. ESEE Report No. 7/89, *Engineering Seismology and Earthquake Engineering, Imperial College, UK.*

Madas, P. and A. S. Elnashai (1992). A New Passive Confinement Model for the Analysis of Concrete Structures Subjected to Cyclic and Transient Loading. *Earthquake Engineering and Structural Dynamics*, **21**, 409-431.

Martinez-Rueda, J. E. and A. S. Elnashai (1995). Confined Concrete Model Under Cyclic Load. Accepted for publication in *RILEM Materials and Structures.*

Salvitti, L. M. and A. S. Elnashai (1995). Evaluation of Behaviour Factors for Frame-Wall RC Structures Designed to Eurocode 8. ESEE Report No. 5/95, *Engineering Seismology and Earthquake Engineering, Imperial College, UK.*