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ASSESSMENT OF EC8 BEHAVIOUR FACTORS FOR RC, STEEL AND COMPOSITE FRAMES

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ABSTRACT

Use is made of advanced analytical techniques for the evaluation of behaviour factors for RC, steel and composite frames. The response parameters used for limit state evaluation are presented alongside comparisons of the use of synthetic and natural earthquake records. Results for one configuration of RC building designed for different ground acceleration and ductility classes as well as a number of different (in height, width, bays and storeys) steel and composite buildings are presented. The combined results indicate that behaviour factors in Eurocode number 8 (EC8) are on the conservative side. Taking into account that the methods and models employed in this study are at worst as valid as those on which existing EC8 behaviour factors are based, it is proposed to increase the existing values for RC, steel and composite structures.

KEYWORDS

Behaviour factors; RC, steel and composite frames; Eurocode 8; inelastic dynamic analysis; limit states.

INTRODUCTION

Debate abounds regarding the values of response modification factors, defined as the ratio between ordinates of the elastic and the inelastic spectra. In the USA, values of R factors are under continuous scrutiny for various materials and structural systems and comparisons between codified values and corresponding New Zealand and European recommendations are undertaken. Analytical assessments of structural response and of strong-motion records have often indicated that US values of response modification are rather optimistic; many US researchers agree that they have been agreed upon by requirements of industry and committee consensus, rather than analytical evidence. It is, however, also true that the low level of damage sustained by US structures in major earthquakes lends an unquantifiable level of weight to the recommended values, which are, on the whole, higher than European values.

Response modification values given in EC8, referred to as q-factors, were derived by extensive analytical studies in the late seventies and early eighties. From that time until the early nineties, only limited effort has been allocated for their re-assessment, since more pressing needs for code development were underway to satisfy the ambitious aim of issuing a code for buildings, bridges, towers, masts, tanks, silos, pipelines, foundations and repair/re-design. In the process of evaluating behaviour factors for a new type of composite steel-concrete building frame, Broderick (1994) studied a number of steel frames, purely for comparison purposes. It became clear that EC8 behaviour factors for steel frames are quite conservative, in addition to the observation that use of steel q-factors for composite frames is completely inappropriate. The latter conclusion from Imperial College remained controversial until recently confirmed by work at the University of Darmstadt (Kantz, 1995). Also in 1994, a large European activity was launched under the auspices of the European Union. The network "Prenormative Research in Support of Eurocode 8" (PREC8) included a collaborative effort in assessing a number of RC structures designed to EC2 (concrete) and EC8 (earthquake resistance). Many, if not all, the results indicated that q-factors were under-estimated in the current version of the code. Moreover, in 1995 the UK Department of the Environment commissioned the primary author to investigate various aspects of the ENV parts of EC8 (Elnashai, 1995). In this study, a number of steel frames, additional to those investigated by Broderick (1994), were analysed. The results also confirmed the conservatism of the EC8 values. Finally, an ongoing study on semi-rigid frames at Imperial College (Elnashai and Elghazouli, 1994) has recently progressed to the evaluation of q-factors for three different configurations of steel frame with partial strength connections. Preliminary results indicate also that q-factors for this type of structure, usually not recommended for seismic applications, may be slightly higher than the values recommended for fully-welded structures in EC8.

The studies quoted above collectively cannot be serendipitous, hence it may be timely to assess EC8 values of behaviour factors, thus leading to more economical design against seismic loading. Below, brief accounts of the studies undertaken at Imperial College are given, and specific observations and recommendations are made.

REINFORCED CONCRETE FRAMES

Description of Structures and Models for Analysis

An 8-storey reinforced concrete structure with a core-frame system is considered and has overall plan dimensions of 15m by 20m, a total height of 24m and equal storey heights of 3.0m. The same general layout is utilised in four distinct design cases. The buildings, which are designed and detailed to EC2 and EC8 (Fardis, 1994), comply to ductility classes "H", "M" and "L", for design ground accelerations of 0.15g and 0.30g, as summarised in Table 1. This is in contrast with the composite and steel frames considered in subsequent sections, most of which are designed for the same ground acceleration.

Table 1. RC Frames 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

The structures are modelled and analysed using the advanced computer code ADAPTIC (Izzuddin and Elnashai, 1989), which has been extensively tested and calibrated elsewhere. Two dimensional analysis is utilised, due to the regularity of the structure both in terms of stiffness and strength. In the direction of loading, the core structure reduces to two RC walls coupled by deep beams at floor levels, whilst the perimeter structural system is a moment-resisting frame. The analysed model comprises therefore of four overlaid 2D frames, coupled appropriately with regard to translational and rotational degrees of freedom. The models were verified and extensively checked in relationship with the analytical results given in the original design document (Salvitti and Elnashai, 1995).

Limit States and Input Motion

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 recorded for top steel at T-beam ends and top and bottom bars for all symmetrically-reinforced sections. A commonly-used global yield definition was utilised, assuming an equivalent elasto-plastic system with reduced stiffness, evaluated as the secant passing through 75% of the maximum load. A critical concrete strain criterion defined by Dodd *et al.* (1994) was also selected following a comparison with that suggested in EC8 in the light of its conservative nature (Salvitti and Elnashai, 1995). Hence, the critical strain (section level ultimate limit state) was defined as that corresponding to a 15% drop in the confined concrete stress on the descending branch of the stress-strain curve. A maximum level of deformation at the global level was defined by a 3% interstorey drift limit and used as the second ultimate limit state criterion.

To preserve consistency and provide bench-marking results for other PREC8 studies, use is made of four accelerograms which fit the EC8 spectrum for soil class B, generated and distributed by the Laboratorio Nacional de Engenharia Civil (LNEC) in Lisbon, Portugal. These are referred to herein as EC8-1 through EC8-4.

Behaviour Factors for RC Frame-Wall Structures

The yield and ultimate limit state peak ground acceleration for the four structures was identified by iterative inelastic dynamic analysis. The confined concrete critical strain (I) and 3% maximum interstorey drift (II) collapse criteria were in turn combined with the first steel yield (i) and equivalent elasto-plastic yield (ii) definitions. Behaviour factors q' and q'_D were evaluated, where the former is defined as the ratio of peak ground accelerations for ultimate and yield limit state satisfaction. The latter definition assumes that yield occurs at a ground acceleration equal to the quotient of the design acceleration and the behaviour factor. A discussion of the two definitions is provided in Broderick and Elnashai (1995a), Elnashai and Broderick (1995) and Salvitti and Elnashai (1995). Herein, it suffices to state that the former is the inherent behaviour factor whilst the latter gives an indication of the conservatism or otherwise of the behaviour factor used in the design of the structure considered. In the context of recommending q-factors for new systems, the former (q') should be used. In assessing existing behaviour factors, the latter value (q'_D) is more appropriate. The results are summarised in Tables 2 and 3.

Table 2. Behaviour Factors for the 0.30g Designs

	DCH030						DCH030					
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

Further consideration of the difference between q' and q'_D is instructive, in the light of the results of Tables 2 and 3. As an example, structure DCM015 is selected. Firstly, minimum values should be considered for safety considerations, hence only values of q' and q'_D of 0.98 and 3.38, respectively, are commented upon. The two q-factors share the same ultimate limit state (I), but differ in the definition of yield. The former evaluates yield from analysis whilst the latter assumes that yield occurs at an acceleration equal to a_D/q, where a_D is the design acceleration. Therefore, if the design was altered, as a consequence of a slight reduction of q (as implied by the value of q') it is likely that both the yield and the ultimate accelerations will increase, with the former being more affected than the latter. The result will be either little or no effect on q'. On the other hand, reducing some member sizes, in sympathy with the value of q'_D, would probably cause a slight increase in q', since the yield state will normally be more affected than the ultimate state.

Whereas this discussion is qualitative, and hence subject to controversy, the further analysis presented by Salvitti and Elnashai (1995) settles the issue. In the latter reference, the structures were subjected to earthquake ground motion up to twice the design acceleration. With the exception of one of four input motion records, the rotational ductility demand in the most stressed coupling beams did not exceed 1.5, whilst a maximum value of 7 was recorded in two coupling beams under twice the design acceleration of only one record. The average of all four motions in the latter case was close to 5. The results for all four frames show similar trends, where the response was perfectly adequate under twice the design event, thus confirming that the relevant response modification factor is q'_D and not q'. From Tables 2 and 3 above, the minimum value of mean q'_D-to-design q-factor is 2.48. It is hence concluded that for the sample structures studied, it is clear that behaviour factors in EC8, at least for coupled frame-wall and core structures, could be increased. Moreover, column ductility demands were very low in all cases. This, coupled with the recorded maximum drift of 2%, confirms that a reduction in column overstrength may be feasible.

STEEL FRAMES

Configuration of Frames

Two separate studies have been undertaken at Imperial College for the evaluation of behaviour factors for steel frames. The first was aimed at comparing q-factors for steel and composite frames (Broderick, 1994; Broderick and Elnashai, 1995a; Elnashai and Broderick, 1995). In that study, ten frames were analysed under six earthquake records. The frames varied from 2x2 (storeysxbays) to 10x5. Spans varied from 5.0 to 8.0m whilst column heights were either 3.0 or 3.6m. All frames were designed to a ground acceleration of 0.25g, in contrast to the RC case, where the same structural configuration was designed to different ground accelerations and ductility classes. The second study, which was funded by the UK Department of the Environment, was aimed at assessing the ENV parts of EC8 (Elnashai, 1995). This included an additional sample of nine steel frames of configuration 2x2 (storeysxbays), 3x5 and 6x3. The frames were designed to accelerations of 0.15g, 0.25g and 0.35g using EC3 (steel) and EC8. The design behaviour factor was 6.0 for all frames.

The analysis environment mentioned above for RC frames was also utilised here. Two dimensional analysis was employed, since the hypothetical frames were regular. The slab was considered to be composite in full interaction with the beam member.

Limit States and Input Motion

A number of limit state criteria were employed in the two aforementioned studies, on the local and global levels. These were as follows:

i. Interstorey drift of 3%
ii. Storey stability index (EC8) of 30%
iii. Collapse mechanism formation
iv. Degradation of lateral resistance by more than 10% for each storey
v. Column rotation ductility limit due to local buckling
vi. Exceedance of concrete slab or steel bottom flange compressive strain in beams

In all cases, the yield condition is the attainment of the yield moment of the section, taking into account moment-thrust interaction. In considering the member-structure response, this coincided in most cases with significant departure from linearity of the frame response.

Table 4. Characteristics of Strong-Motion Records Used in Analysis

Record	Epicentral Distance (km)	Soil Type	Magnitude M_L	Peak Ground Acceleration (g)	Peak Ground Velocity (m/s)	a/v Ratio $g/(ms^{-1})$	Period of Max. Amplification (s)
Friuli	52	Rock	6.4	0.159	0.080	1.99	0.95
Gazli	14	Int.Stiff.	7.3	0.724	0.606	1.20	0.13
L. Prieta EW	97	Soft	7.1	0.213	0.216	0.99	0.65
El Centro	8	Stiff	6.6	0.344	0.365	0.94	0.26
Spitak	27	Int.Stiff.	6.8	0.182	0.237	0.77	0.36
L. Prieta NS	97	Soft	7.1	0.250	0.433	0.58	1.20

Again in contrast with the study on RC frames, this work employs a number of natural earthquake records selected to cover the range of a/v, where a and v are the peak ground acceleration and velocity, respectively. This parameter was shown to be sensitive to many engineering seismology characteristics of strong-motion, such as frequency content, distance from source, duration and site condition. Records selected to cover low, medium and high a/v will embrace a number of strong-motion characteristics, rendering the selection feasible for representative earthquake analysis. The characteristics of the six records employed in the analysis, which are also those utilised for composite frames, are given in Table 4 above.

The above set of records has been used in previous studies on RC buildings and bridges, as well as steel semi-rigid frames. With the exception of the Friuli record, which is characterised by a very narrow band frequency content, results have been shown to have an acceptable dispersion.

Calculated Behaviour Factors

In addition to definitions used for the evaluation of behaviour factors presented for RC frames, as above, other definitions exist, offering various advantages in seismic assessment. For instance, it is usual that a_y (ground acceleration corresponding to yield limit state) is distinct but close to that implied by design ($a_{yD} = a_D/q$). On the other hand, use of a_D/q as the acceleration corresponding to yield limit state, as performed for RC structures above, obscures the strong-motion dependence of q-factors. If the yield acceleration is scaled in such a way as to account for the design overstrength, whilst maintaining the design-implied value (i.e. $a'_y = a_y/a_{yD}$), this could be a powerful tool for assessment of the overstrength of the structure, since a'_y should be equal to unity. Furthermore, correspondence to the ultimate limit states may also be drawn with values at yield, hence arriving at a number of q-factor definitions that include for both strong-motion dependence and the design-implied response. An example of the latter would be to use the design drift of 1.5% to scale the yield acceleration (i.e. $a'_{ydrift} = a_D[1.5/\delta_D]$, where δ_D is the drift under the design event). The drift-based behaviour factor is then given as $q'_{drift} = a_{gdrift}/a'_{ydrift}$. The above definitions were considered in the study of steel frames, and minimum values of q' and q'_D for all frames are shown in Table 5.

Table 5. Minimum Behaviour Factors q' and q'_D for Steel Frames

No. of Storeys	No. of Bays	Storey Height (m)	Bay Width (m)	Live Load (kN/m^2)	Design Accn. (g)	q'	Critical Input Motion	q'_D	Critical Input Motion
2	2	3.6	8.0	3.5	0.25	9.10	Friuli	7.38	El Centro
2	2	3.0	8.0	3.5	0.25	8.98	Friuli	8.22	Spitak
3	3	3.6	8.0	3.5	0.25	9.27	Friuli	9.36	El Centro
3	3	3.0	8.0	3.5	0.25	9.32	Gazli	11.10	Spitak
3	5	3.0	8.0	3.5	0.25	10.15	L.Prieta NS	9.72	Spitak
3	5	3.0	5.0	5.5	0.25	9.51	Gazli	9.18	Spitak
6	3	3.0	8.0	3.5	0.25	11.58	L.Prieta NS	10.62	L.Prieta EW
6	3	3.0	8.0	5.5	0.25	10.93	Spitak	10.86	L.Prieta NS
6	5	3.0	8.0	3.5	0.25	11.34	Friuli	10.92	L.Prieta NS
10	5	3.0	8.0	3.5	0.25	12.49	Friuli	11.64	L.Prieta NS
2	2	3.6	8.0	3.5	0.15	1.63	L.Prieta EW	12.80	Spitak
2	2	3.6	8.0	3.5	0.25	1.63	L.Prieta EW	7.68	Spitak
2	2	3.6	8.0	3.5	0.35	1.63	L.Prieta EW	5.49	Spitak
3	5	3.6	5.0	3.5	0.15	3.30	Spitak	13.20	Spitak
3	5	3.6	5.0	3.5	0.25	3.30	Spitak	7.92	Spitak
3	5	3.6	5.0	3.5	0.35	3.83	Spitak	7.89	Spitak
6	3	3.6	5.0	3.5	0.15	4.36	Spitak	9.60	Spitak
6	3	3.6	5.0	3.5	0.25	5.00	Friuli	8.40	Spitak
6	3	3.6	5.0	3.5	0.35	5.26	Friuli	8.57	L.Prieta EW

The results given in Table 5 indicate clearly that EC8-defined behaviour factors could be safely increased, with minimum value of mean q'_D-to-design q-factor being 1.25 in only one case, whilst the average is 1.65. Since these values are derived from drift limits, it follows that other criteria have not been satisfied, hence the frames have not been subjected to excessive ductility demand. It is also noteworthy that this study incorporated a two-stage scaling technique (first to the target ground acceleration then to a target velocity spectral intensity) which is inherently over-conservative, a comment in support of the suggestion to use higher q-factors in EC8.

An interesting issue arises from consideration of the drift control approach used in EC8, where the allowable drift of 1.5% is checked versus the product of the elastic displacement and the q-factor. No benefit will ensue from increasing q, since this will be linearly reflected in the drift check, which by-and-large controls the design. It is therefore recommended that the limitation on drift is reviewed alongside the q-factors. Pertinent to this discussion is to reiterate that at 3% drift, imposed ductility demand was well below capacity.

COMPOSITE FRAMES

Work at Imperial College has concentrated on composite frames for a number of years, staring with on-line tests on composite members (Elnashai *et al.*, 1991) and frames (Takanashi *et al.*, 1992). Analytical studies followed, leading to design recommendations (Broderick, 1994; Broderick and Elnashai, 1995b). In the latter studies, ten frames were extensively analysed using six natural earthquake records. Frame configurations and input motion characteristics are similar to those described for steel frames above. All limit state definitions were conceptually similar to those used for steel frames. An analytical expression for local buckling of partially encased composite members, similar to that defined for steel by Kato (1989), did not exist. A criterion which accounts for the presence of concrete and its effect on the equivalent section and the critical buckling strain was derived specifically for this study (Broderick, 1994) and employed to signal the attainment of the member rotation ductility supply.

Calculated Behaviour Factors

The discussion presented for steel frames above pertains herein. Values of scaled q-factors, representing q' and q'_D are given in Table 6. It is again evident that an increase in EC8 behaviour factors is warranted. Assessment of the critical states in the frames considered confirms that ductility demand was not excessive and that the frames possessed adequate structural integrity at ground accelerations well above the design values. This, in tandem with the rather conservative two-stage strong-motion scaling procedure used, lends weight to the proposal for increased q-factors for composite frames. The design acceleration was 0.25g for all frames.

Table 6. Minimum Behaviour Factors q' and q'_D for Composite Frames

No. of Storeys	No. of Bays	Storey Height (m)	Bay Width (m)	Live Load (kN/m^2)	q'	Critical Input Motion	q'_D	Critical Input Motion
2	2	3.6	8.0	3.5	8.77	Friuli	9.60	Spitak
2	2	3.0	8.0	3.5	9.08	Friuli	8.46	Spitak
3	3	3.6	8.0	3.5	8.48	Friuli	7.92	Spitak
3	3	3.0	8.0	3.5	9.90	Gazli	9.12	Spitak
3	5	3.0	8.0	3.5	9.84	L.Prieta NS	7.92	Spitak
3	5	3.0	5.0	5.5	10.00	Friuli	7.26	Spitak
6	3	3.0	8.0	3.5	10.44	Gazli	10.38	L.Prieta NS
6	3	3.0	8.0	5.5	11.72	Spitak	10.50	L.Prieta NS
6	5	3.0	8.0	3.5	11.28	Friuli	10.68	L.Prieta NS
10	5	3.0	8.0	3.5	12.22	Spitak	11.46	L.Prieta NS

Various interesting issues emanate from the study of composite frames with regard to the contribution to lateral stiffness from beams and columns and the importance of beam response in both positive and negative bending. These issues are addressed elsewhere (Broderick and Elnashai, 1995a; Elnashai and Broderick, 1995). Herein, of most relevance to the thesis of this paper is the comparison between steel and composite frames. With regard to q-factors, the difference between the two materials is negligible. However, for structural systems other than moment-resisting frames, the presence of concrete may alter the failure mode, thus causing a serious erosion in the action re-distribution potential of composite frames (Kantz, 1995). It is therefore essential that studies specific to composite structures are used to recommend q-factors. The argument that concrete can only be beneficial, hence use of steel q-factors for composite frames is conservative, exhibit conceptual shortcomings.

CONCLUSIONS

Subject to the approximations inherent in the studies presented above, and limited to the structural forms investigated, the results presented give a clear indication that behaviour factors in EC8 may be safely increased. Moreover, for RC structures where the columns are not critical, such as core and frame-wall systems, it is likely that a reduction in column overdesign factors will not adversely affect the seismic response to levels well above the design event. Finally, the results for steel and composite frames demonstrate that for moment-resisting systems, behaviour factors for the two construction types are rather similar. This may not, though, be the case for braced frames, and special studies are urgently needed.

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