MODERN SEISMIC DESIGN OF STEEL STRUCTURES AND EC8 DEVELOPMENTS

Amr S ELNASHAI[1]

SUMMARY

Recent earthquakes have lead to the conclusion that the inherent seismic resistance of steel structures should not be taken for granted. Since the damage inflicted on connections in Northridge, concerted efforts in several countries have lead to serious modifications of design procedures and material control measures as well as welding practice. Another development that influenced seismic design of steel is the progress made in displacement-based design and assessment procedures alongside the generalisation of limit state concepts into a performance-based design framework. In this paper, recent findings on the material, section, member and sub-assemblage levels are reviewed and possible code applications are highlighted. Also, investigations on steel frames with bare and composite slabs are reviewed and distilled. The requirements for the revision of EC8, in view of its conversion from ENV to EN, are listed and matched with the developments previously mentioned. The paper is complementary to that presented in this session by Professor A.Plumier on composite steel-concrete structures.

INTRODUCTION

Steel structures constitute a substantial proportion of construction in Europe. A comprehensive survey of design and construction projects in Europe [Elnashai et al, 1995] indicated that in fact steel is in the majority, albeit by a small margin, as shown in Figure 1. It is therefore essential that European design codes (Eurocodes 2 and 8) continue to be at the forefront of developments.

For many years, steel structures were viewed as the ideal form of construction for seismic resistance, especially in areas of high seismic hazard [AISI, 1991]. This was manifested in codes giving minimum requirements for design and detailing, as compared to sections dealing with reinforced concrete, which are usually voluminous. This belief was undermined by the damage inflicted on various types of steel structure in the Northridge [Southern California, 1994] and the Hyogo-ken [Kansai, 1995] earthquakes. Various publications give accounts of the extensive damage inflicted on steel structures, particularly though not exclusively, frames [EERI, 1994, AIJ, 1994, Broderick et al, 1994, Elnashai, 1994, Elnashai et al, 1995]. This upheaval was approximately coincident with the drafting of the final parts of Eurocode 8, which was issued in 1995. It became apparent to those involved that major revisions will soon be needed, to bring the code up to date, and to take stock of the observations and subsequent remedial action. This is also aided by the extensive developments currently underway to draft a additional chapter in Eurocode 8 on composite structures. These developments are a subject of another paper in this volume.

[1] *Professor & Head of Section, Earthquake Engineering, Civil Engineering Dept., Imperial College, London SW7 2BU*

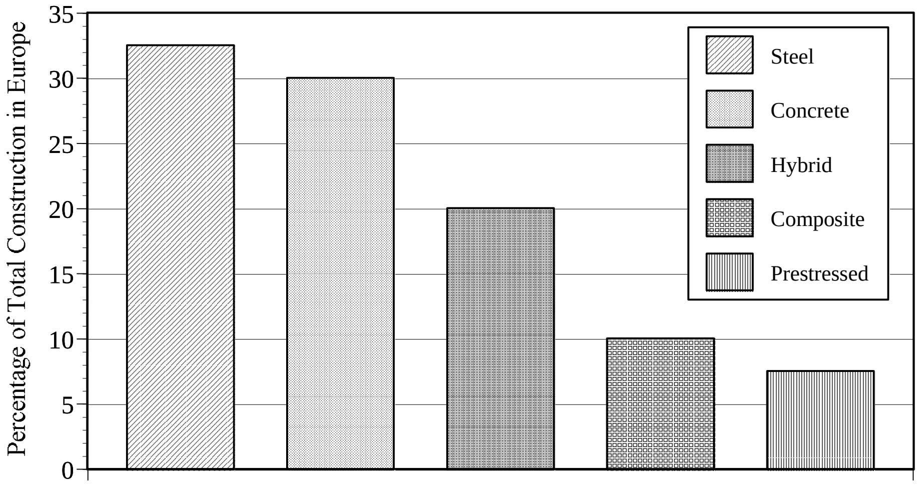

Figure 1: Structural materials in design and construction projects in Europe [Elnashai et al, 1995]

It is the objective of this short paper to highlight the areas of European and international development that may be amenable to codification and inclusion in Eurocode 8. It is emphasised that it is not a review of all major developments by all research groups in Europe. A useful reference where many recent developments are described is that by Mazzolani and Piluzo [1996].

MATERIAL CHARACTERISTICS

It is clear that the ratio between ultimate stress and yield stress has a significant effect on the length of zone of inelasticity (plastic hinge length). The higher this stress ratio is (defined as fu/fy, where the former is at ultimate and the latter at yield), the more will inelasticity spread in the member. The consequence being that higher displacement ductility will be achieved for a given curvature ductility, or lower curvature ductility will be required to achieve a given displacement ductility. Coupon tests from British Steel were collected and analysed (Manzocchi and Elnashai, 1994) showing that the variability of yield stress is sufficiently high so as to undermine capacity design objectives for steel frames. Work progressed towards deriving contour plots with the coefficient of variation in yield stress of beam and column as ordinates and abscissae, with the contour lines giving frame ductility with a given level of confidence [Manzocchi et al, 1995].

The issue of strain rate has been a controversial one for some years. It was postulated that dynamic material properties are irrelevant to the medium frequency range typical of earthquake loading. This may be true in most cases. However, where buckling of members is a possibility, high strain rate situations may arise, even under static loading [Elnashai and Izzuddin, 1993]. It may therefore be prudent in seismic design codes to give guidance on situation where energy dissipation in yielding members would not materialise due to unexpected increases in yield strength.

SECTION DUCTILITY

Slenderness restrictions are imposed on sections to provide minimum curvature ductility, normally governed by local buckling. Traditionally, the slenderness requirements are imposed on webs and flanges separately. Kato [1989] developed a procedure for imposing restrictions on both flange and web concurrently through an interaction formula. In order that a target curvature ductility is reached, the geometry of flange and web should satisfy the condition given by equation 1, namely:

$$\frac{\left(b / t_f\right)^2}{\left(E / 1.6 f_{fy}\right)\left(1 / s - 0.6003\right)} + \frac{\left(d_e / t_w\right)^2}{\left(E / 0.1535 f_{wy}\right)\left(1 / s - 0.6003\right)} = 1.0 \tag{1}$$

This procedure was extended to the case of steel members partially encased in concrete by Broderick and Elnashai [1994]. The web-flange interaction idea of Kato is a more appropriate condition than separate control of flange and web, since the two components provide mutual restraint. It is therefore recommended that EC8 adopts a similar approach.

CAPACITY, STABILITY AND DUCTILITY OF MEMBERS

Effective Width of Slab

Considerable work has been undertaken on beams in recent years. As the members mainly responsible for energy dissipation, in a capacity-design framework, beams may be subjected to excessive ductility demand, both in positive and negative bending [Plumier et al, 1998]. One of the areas of serious developments has been the definition of the effective width of slab contributing to beam response. It is clear that over-estimating the effective slab width would lead to optimistic estimates of capacity, whilst under-estimating this would lead to undermining of capacity-design of columns. It is equally clear that the effective width is not constant and is a function of, amongst other factors, the deflection of the beam. Recent work by Migiakis and Elghazouli [1998] has indicated that the increase in beam stiffness and strength due to an increase in effective slab width, in comparison with code-recommended values, is in excess of 20%. This is particularly noticeable for long beam lengths. The work is, however, not sufficiently developed to allow for a proposal for code implementation. This is an area of considerable current interest.

Shear Connection

Whereas slip at the concrete slab-steel beam interface is an effective source of energy dissipation, the ensuing deformations may cause the early attainment of a limit state, thus hindering the available deformation capacity. Giving the designer a range of options for section ductility and dissipation through slip, is a positive move. Work undertaken by Tsujii and Elnashai [1998] on the effect of shear connection on ductility has indicated that high deformation capacity is achieved by increasing the shear connection well beyond the level needed for full section moment capacity (above 100% shear connection). The ductility of the frame drops very significantly for shear connection ratios below 70%. However, ongoing work by Bursi and co-workers [1999] indicate that low levels of shear connection lead to fatigue failure of the shear studs, hence values below 70% are not recommended.

Effective Length of Column

The concept of an effective length, in relation to the Euler buckling length, has been used to define equivalent Euler struts for stability checks. It is customary to consider the effective length of a column as 0.7 the actual length, due to the connection restraint. When limited yielding in connections is allowed or where semi-rigid connection design is permitted, this constraint is significantly reduced. The consequence is that stability checks undertaken using the conventional values will be unsafe. Guidance is therefore needed to evaluate the effective column length taking into account the state of deformation on the connection and the level of ductility reached there. Tests on bolted top and seat angle web cleated connections [Takanashi et al, 1992, Elnashai et al, 1998] gave some early results for the effective length multiplier. Further work is required to parameterise this effect in terms of connection deformation and other salient parameters. However, conservative estimates could be included in EC8 to safeguard against uninformed application of existing rigid connection guidance.

FRAME BEHAVIOUR AND DESIGN

Application of Semi-rigid Connections

Since the tests of Nader and Astaneh [1988], several studies have been conducted to progress the potential use of bolted semi-rigid connections in areas of medium or even high seismic hazard. In Europe, early studies [Bernuzzi et al, 1991, 1992] showed that semi-rigid connections posses sufficient cyclic ductility. Later, a testing programme was undertaken at Imperial College, with collaboration with the Institute of Industrial Science, Tokyo [Takanashi et al, 1992, Elnashai et al, 1992, Elnashai et al, 1998]. Twelve nearly full scale models of two storey frames were tested cyclically and using online computer-controlled dynamic procedures. Based on the combined experimental and analytical observations, recommendations regarding the minimum moment capacity of the connection as a percentage of beam capacity were given, as shown in Table 1.

Table 1: Recommended values for moment capacity of connections in frames [Elnashai et al, 1998]

Number of storeys	Strength/Type of Connection	Connection Moment Capacity
Two Storeys	Partial Strength Semi-rigid	30%-50%
Three-Four Storeys	Partial Strength Semi-rigid	50%-70%
Five-Six Storeys	Partial/Full Strength Semi-rigid	70%-100%

In the above, no restrictions would be imposed on such designs with regard to the level of hazard. It is therefore proposed that semi-rigidly connected frames are allowed in earthquake design and that the above guidelines for their capacity are adopted in EC8.

Eccentrically Braced Frames

Very few studies have been conducted on this form of construction in Europe. Therefore, European recommendations on this type of frame is likely to be almost entirely based on US practice [Hjelmstadt and Popov, 1988].

Response Modification Factors

Considerable European activities have been devoted to the refinement of response modification factors (q or R). Analytical studies have been undertaken [Elnashai et al, 1996, Plumier and Sanchez, 1999, Bouwkamp and Parung, 1999] as well as experiments [Elnashai and Tsujii, 1999]. The latter study constituted an attempt at quantifying the yield and ultimate limit states experimentally by using extensive analytical studies beforehand to narrowly bracket the earthquake scenarios corresponding to the two limit states. A single shaking table run is then attempted, to avoid cyclic degradation due to repeated testing. The dimensions of the test specimen and a photograph on the shaking table are shown in Figure 2, whilst summary results are given in Table 2, where 'sb' and 'wb' refer to strong or weak beam, relative to column.

Table 2: Experimental behaviour factors [Elnashai and Tsujii, 1999]

Frame	a_{gy}	a_{gu} (g)	δ_y (mm)	δ_u (mm)	μ	q
comp. 1 (sb)	0.225	1.22	39	103	2.6	5.4
comp. 2 (wb)	0.062	0.56	26	98	3.7	9.0
steel 1 (sb)	0.226	1.21	43	102	2.4	5.3
steel 1 (wb)	0.058	0.53	34	94	2.7	9.1

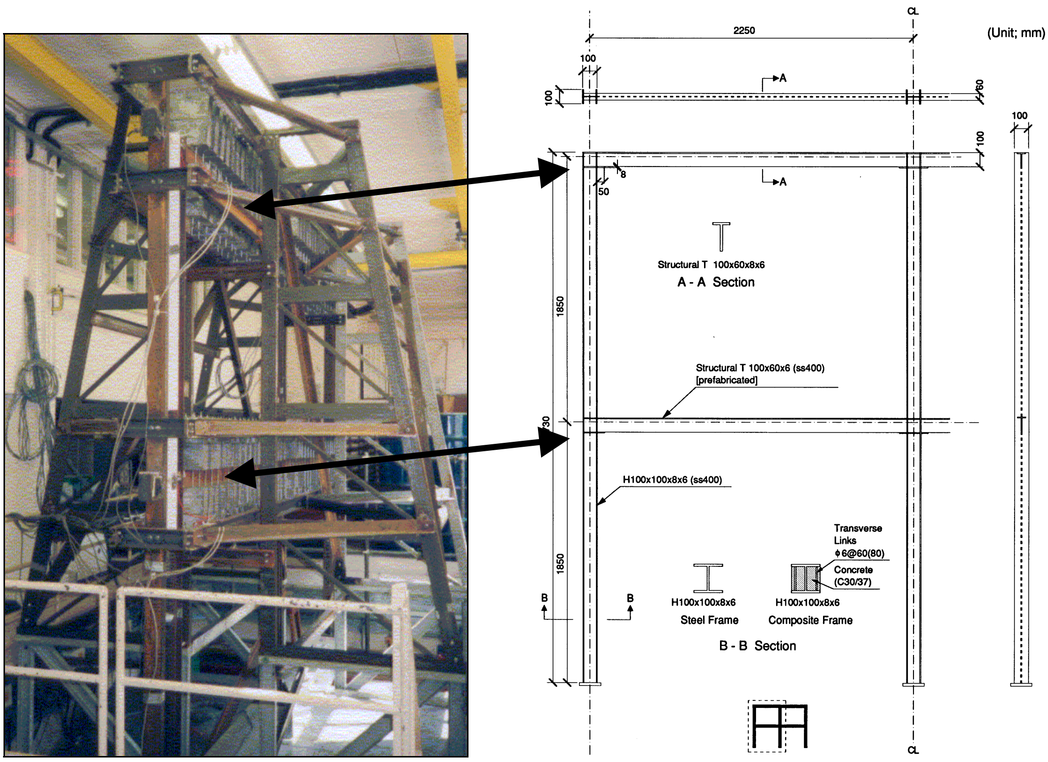

Figure 2: Dimensions of test frame (right) and photograph of frame in the Bristol Table (left)

In the light of the large amount of evidence compiled, it is likely that there will be a slight increase in behaviour factors. This will still be linked to over-strength since the design behaviour factor is clearly affected by the ratio of design-to-required load resistance.

CONCLUDING REMARKS

Significant developments have taken place both in Europe and world-wide in seismic design of steel structures. This has been clearly accelerated by the unexpected severity of damage inflicted on steel structures in two recent large earthquakes in the USA and Japan. On the other hand, Eurocode 8 is currently undergoing a procedure of conversion from ENV (European pre-norm) to EN (full European standard). The project team entrusted with this conversion is taking stock of recent development to make EC8 as modern and progressive as possible, whilst maintaining a consistent level of safety and simplicity. Above, some of the many proposed changes are briefly presented, and further work is underway. Considerable work has been undertaken within European research networks, such as RECOS, PREC8, ICONS, ECOEST and others, and more is planned. Is effort is underpinned by funding from the EU, and is justified by the observation that steel structures constitute a narrow majority in construction in Europe.

References

AIJ [1995] *Preliminary Reconnaissance Report of the 1995 Hyogoken-Nanbu Earthquake*, Architectural Institute of Japan, April, 216 pp.

American Iron and Steel Institute [1991] *Performance of steel buildings in past earthquakes*, AISI, 97 pp.

Bernuzzi, C., Zandonini, R. and Zanon, P. [1991] “Rotational behaviour of end plate connections,” *Costruzioni Metalliche*, no. 2, pp. 74-103.

Bernuzzi, C., Zandonini, R. and Zanon, P. [1992] “Semi-rigid steel connections under cyclic loads,” *Proceedings of the First World Conference of Constructional Steel Design*, Acapulco, Mexico, pp. 375-384.

Bouwkamp, J. and Paring, H. [1998] “Evaluation of behaviour factors for moment resisting frames based on the Ispra tests,” *ICONS Topic 4 report*, Madrid, May.

Broderick, B.M. and Elnashai, A.S. [1994] “Seismic resistance of composite beam-columns in multi-storey structures, Part II: Analytical model and discussion of results,” *Journal of Constructional Steel Research*, Vol. 30, pp.231-258.

Broderick, B.M., Elnashai, A.S., Ambraseys, N.N., Barr, J., Goodfellow, R. and Higazy, M. [1994] “The Northridge (California) earthquake of 17 October 1994; observations, strong-motion and correlative response analyses,” *Engineering Seismology and Earthquake Engineering Report No. ESEE 4/94*.

Bursi, O., 1999, Private communication.

EERI [1995] “The Northridge Earthquake Reconnaissance Report, Earthquake Spectra Supplement C to vol. 11,” *Earthquake Engineering Research Institute*, April, Vol. 1 523 pp., Vol. 2 278 pp.

Elnashai, A.S. [1994] “Comments on the performance of steel structures in the Northridge (Southern California) earthquake of 17 January 1994,” *New Steel Construction*, Vol. 2, No. 5, October.

Elnashai, A.S., Danesh-Ashtiani, F.A. and Elghazouli, A.Y. [1998] “Response of semi-rigid steel frames to cyclic and earthquake loads,” *ASCE, ST Division*, Vol. 124, No. 8, pp.857-867.

Elnashai, A.S., Bommer, J.J., Baron, C.I., Lee, D., Salama, A.I. [1995] “The Hyogo-ken Nanbu Earthquake of 17 January 1995, Selected Engineering Seismology and Structural Studies,” *Engineering Seismology and Earthquake Engineering Report No. ESEE 2/95*, September.

Elnashai, A.S., Broderick, B.M. and Burgan, B.A. [1995] “State-of-the-practice in the use of steel in European seismic design, European Seismic Design Practice,” *5th SECED conference*, Chester, pp. 229-236.

Elnashai, A.S., Broderick, B.M. and Salvitti, L.M. [1996] "Assessment of EC8 Behaviour Factors for RC, Steel and Composite Frames," *Eleventh World Conference on Earthquake Engineering*, June 23-28, Acapulco, Paper No. 2050.

Elnashai, A.S. and Chryssanthopoulos, M. [1991] "Effect of material variability on response of steel frames," *Earthquake Engineering and Structural Dynamics*, Vol. 20, January.

Elnashai, A.S. and Izzuddin, B.A. [1993] "Sources of uncertainty and future research requirements for seismic analysis of structures," *Nuclear Engineering*, Vol. 32, No. 2, pp.213-220, August.

Elnashai, A.S., Takanashi, K., Elghazouli, A.Y. and Ohi, K. [1992] "Experimental behaviour of steel and composite frames under cyclic and dynamic loading," *Engineering Seismology and Earthquake Engineering Report No. ESEE 10/92*, November.

Elnashai, A.S. and Tsujii, M. [1999] "Shaking table tests and analysis of steel and composite frames," *ICONS Topic 4 report*, Ispra, Italy, June.

Hjelmstadt, K.D. and Popov, E.P. [1983] " Seismic behaviour of active beam links in eccentrically braced frames," *EERC Report No. UBC/EERC-83/15*, University of California, Berkeley.

Izzuddin, B.A. and Elnashai, A.S. [1993] "The influence of rate-sensitivity on the response of steel frames," *Eurodyn 93*, Trondheim, pp.617-624, 21-23 June.

Kato, B. [1989] "Rotational capacity of H-section members as determined by local buckling," *Journal of Constructional Steel Research*, Vol. 13, pp. 95-109.

Manzocchi, G.M.E., Chryssanthopoulos, M. and Elnashai, A.S. [1995] "The influence of material variability on failure mode control of steel frames," *European Seismic Design Practice, 5th SECED conference*, Chester, pp. 245-251.

Manzocchi, G.M. and Elnashai, A.S. [1994] "Characteristics of steel in the United Kingdom. Prenormative research in support of Eurocode," *Report No. PREC 8-01*, May.

Mazzolani, F.M. and Piluso, V. [1996] *Theory and design of seismic resistant steel frames*, E &FN Spon, London.

Migiakis, C.E., and Elghazouli, A.Y. [1998] "Slab effect in composite floor systems subjected to earthquake loading," *Proceedings of the 3rd National Conference on Steel Structures*, Thessaloniki, Greece, pp. 328-336.

Nader, M.N. and Astaneh, A. [1988] "Behaviour and design of steel Tee-framing connections," *University of California at Berkeley*, *Report no. UCB/SEMM-88/11*.

Plumier, A., Doneux, C. and Bouwkamp, J. [1998] "Slab design in connection zones of composite frames," *11th European Conference on Earthquake Engineering*, Paris, September.

Plumier, A. and Sanchez, L. [1999] "Re-assessment of behaviour factors for steel and composite frames," *ICONS Topic 4 report*, Ispra, Italy, June.

Takanashi, K., Elnashai, A.S., Elghazouli, A.Y. and Ohi, K. [1992] "Seismic response of steel frames with semi-rigid connections," *Constructional Steel Design: World Developments. First World Conference on Constructional Steel Design*, Acapulco, December, Elsevier Science.

Tsujii, M. and Elnashai, A.S. [1998] "Effect of shear connection in composite slabs on energy dissipation capacity and behaviour factors of frames," *ICONS Topic 4 Extended Group Report*, Madrid, May.