Earthquake Engineering, Tenth World Conference © 1992 Balkema, Rotterdam. ISBN 90 5410 060 5

Comparison of R/C and composite frames for earthquake resistant design

Jack G. Bouwkamp
Technical University, Darmstadt, Germany
Ekkehard Fehling
Bouwkamp, Fehling and Partner, Consulting Engineers, Darmstadt, Germany

ABSTRACT: In order to compare reinforced concrete and composite frame structures for use in earthquake prone regions the design of two frames are presented. The design examples are basically similar to the composite frames tested in full scale at the Technical University of Darmstadt. Specific observations related to the design calculations and details of both alternatives are presented, including recommendations. Since the current drafts of EUROCODE 8 (earthquake design) together with EUROCODES 2,3 and 4 (for concrete, steel and composite design, respectively) were used as the basis for the design checks, practical experience with the use of the Eurocodes could be gathered. The results of this comparative study identified the needs for both future research and code modifications.

1 INTRODUCTION

Composite structures can be regarded as viable design alternatives for both reinforced concrete and steel structures. In composite design the advantages of both materials can be exploited. In the last 20 years research and practice in western Europe have proven the usefulness of composite construction. Because of prefabrication and ease of erection on the one hand and the simplicity of installing interior and exterior nonstructural elements on the other hand, composite construction has become an increasingly interesting alternative, for both industrial and commercial buildings. Based on the results of extensive fire-safety research, composite construction has been accepted as a structural system with an inherently excellent fire resistance. However, because such systems had not been built so far to withstand serious earthquake exposure, an extensive cooperative experimental research program funded by the European Convention for Construction Steelwork (ECCS) and managed by ARBED-Research, Luxembourg, was started in 1987. First results of experimental studies performed by Ballio & Plumier (1990) and Bouwkamp (1991), clearly support the feasibility of using composite construction in earthquake prone regions. Further experimental studies aimed at optimizing the design of composite-frame systems including eccentrically-braced frames are presently pursued. In addition to experimental studies, also actual design feasibility studies incorporating both test results and code provisions are underway to guide the overall research effort and assess present code provisions. The paper presented herewith covers the findings of such a feasibility study.

2 DUCTILE COMPOSITE FRAME STRUCTURES - RESEARCH AND CODES -

In Darmstadt a series of tests on composite frames with both bolted and welded beam-column connections has been performed showing an excellent ductile response. The current practice in Europe involves the use of bolted beam-column connections. Steel beam and column sections are typically designed with concrete being placed between the flanges. In order to enhance the beam design the composite beam sections are connected to the concrete floor slabs through welded headed shear studs. Reflecting the above construction practice, the design studies presented in this paper are specifically focussed on this system.

In order to assess the economy of composite frame construction, not only a composite frame, but also a reinforced concrete frame has been designed as an alternative. In the design checks and detailing current drafts of Eurocodes EC 8, Part 1, as well as EC 2 (Reinforced Concrete Structures) and EC 4 (Composite Structures) have been followed. In accordance with the design-check format used throughout the Eurocodes, checks for the serviceabilitylimit-state, the ultimate-limit state, the case of fire, and the case of earthquake loading have been performed. In these design checks the concept of partial safety factors as defined by the Eurocodes has been maintained. In case of earthquake loading the 'Capacity Design' philosophy has been considered. Hence, by introducing additional partial safety factors one has to ascertain that failure of elements or connections will not occur before attaining sufficient ductility. Also, shear failure should not govern the behaviour and should occur only after a condition of ductile bending has been attained.

3 BASIC CONDITIONS FOR COMPARISON

In order to have a realistic earthquake exposure, the ground accelerations as they would have to be considered in Greece, as a member-country of the European Community, have been used to define the seismic design loads. The level of ground acceleration is selected such, that only very little probability of exceedance has to be expected in continental Greece. According to the latest greek seismic code only for western Greece (Patras Region and Ionian islands) a higher level of ground acceleration would have to be considered.

The special design conditions for the two frames are:

- fire-resistance class R 90 (90 minutes),
- strong earthquake (Greece), (design response spectra shown in Fig. 1),
- use of building: warehouse with live load of 5 kN/m^2,
- 4 storeys (storey height 3.50 m typical),
- 3 bays at 9.00 m (c.o.c. columns),
- distance between frames 6.00 m,
- frames loaded in-plane only,
- Reinforced-Concrete-alternative (in situ concrete),
- Composite-alternative (suited for prefabrication and erection; connections with beam endplates, steel sections with concrete filled-in chambers).

The calculation assumes a highly regular building which is stiffened laterally in the perpendicular direction by a system other than frames (no bi-axial bending in columns or overall torsional system response).

4 SPECIFIC OBSERVATIONS ON R/C-FRAME DESIGN

The overall frame plan and member dimensions for the reinforced concrete frame are presented in Fig. 2.

Based on the calculations and design of both the reinforced-concrete and composite steel/concrete frame systems a number of questions related to the Eurocode provisions did arise and have been adressed in the design process of both frames.

Regarding the rules for the design of beam-column joints, as stipulated in EC 8, some questions remain. In order to avoid capacity problems in the beam-column-joint regions the width of the interior columns was selected to be larger than the width of the webs of the T-beam sections. This geometry, simplifies the placement of the longitudinal rebars in the beam-column intersections.

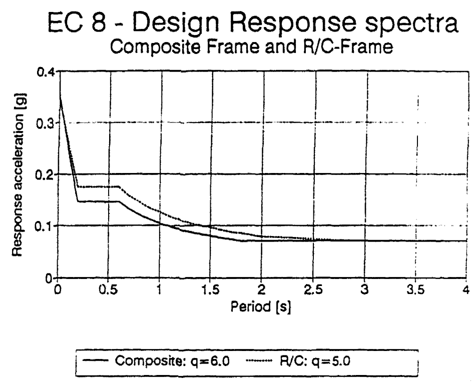

Fig. 1 Design response spectra according to EC 8

In determining the shear capacity of beams and columns, EC 2 allows two possibile procedures for checking the design of beam and column sections and their shear reinforcement (i.e. standard method and variable strut inclination method). In accordance with the pertinent rules given in EC 2, the most economic result of either method has been used in the design calculations. However, from a more scientific point of view this approach does not seem to be justified for several special cases. Because of the severe seismic loading conditions, a relatively high amount of reinforcement is necessary which makes the concreting relatively difficult and time-consuming; a factor which has to be considered when comparing the economic aspects of the two structural systems.

Extremely high and totally unpractical reinforcement ratios can be avoided by following the design and detailing rules given in the RC chapter of EC 8 for members of high ductility (ductility class 'H'; behaviour-factor q = 5.0). These rules necessitate the choice of relatively large concrete cross sections for both beams and columns. Hence, for the RC-frame column-dimensions of 50 * 50 cm (exterior columns)

R/C Frame

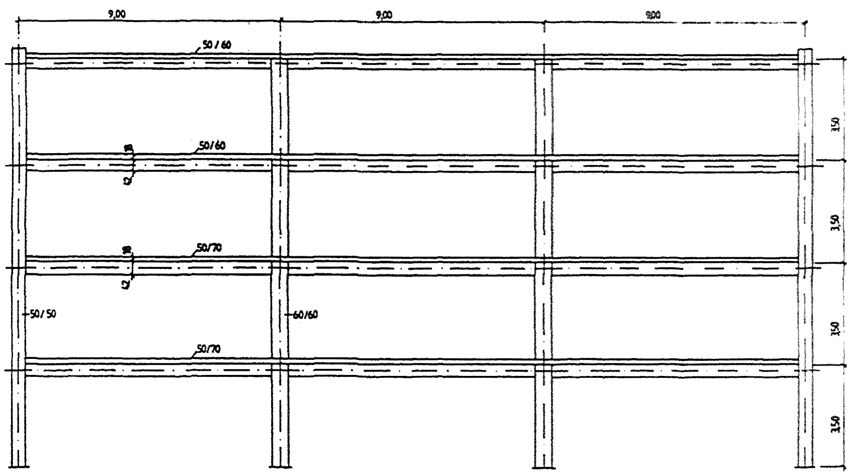

Fig. 2 Reinforced-concrete-frame design

and 60 * 60 cm (interior columns) are required. These values are rather large in comparison with the 30 * 30 cm and 30 * 40 cm dimensions for the corresponding composite columns. The same relation applies to the cross-sectional dimensions of the beams for both alternatives.

While the above considerations may offer certain design choices, other conditions leave the designer without any alternative. The rules for confining hoops in regions where potential plastic hinges may occur, lead invariably to an extremely high number of closely spaced hoops. Also, the code provisions for the anchoring of longitudinal bars exiting or entering exterior columns lead to extreme anchoring lengths and pertinent detailing problems. This consequently leads to the fact, that in many cross sections the number or rebars present is significantly larger than the number required for meeting the cross sectional design resistance. In turn this leads to increasing design demands for sections which are to be designed to remain elastic. Due to the capacity design philosophy the effort of assessing the flexural cross-sectional capacity of beams and columns is almost double to that required typically in the design process of buildings in nonseismic regions. The main reason for this difference is caused by the cross-sectional complexity resulting from the code requirement of placing a minimum amount of reinforcement in the compression zone of flexural members. This in itself leads to a remarkable capacity of flexural members when subjected to reversed moments (i.e.causing tension in the original compresion zone); this information has to be quantified before calculating the capacity-design forces for the columns.

5 SPECIFIC OBSERVATIONS ON COMPOSITE-FRAME DESIGN

For the design checks of the composite frame both Eurocodes EC 8 and EC 4 (Composite Structures) have been used. Similar to the full-scale frame tests performed at the Technical University, Darmstadt, in the first design study HEB-sections were selected used for the columns and HEA-sections for the beams (see Fig. 3). Specifically, the interior column sections were designed as HEB 400 for the ground floor column, HEB 340 for the following two floors and HEB 300 for the top floor. The beam sections selected were HEA 400 for the first three floors and HEA 300 for the roof. In order to optimize the first design, a redesign in which the beams were to be replaced by IPE sections and the columns by HEA sections has been carried out. The resulting design is shown in Fig. 4. The steel used in both designs was St 37 (Fy = 34 ksi).The composite frame has been designed with the same floor slab thickness of 18 cm as the RC frame.

In general, columns and beams are prefabricated composite elements with infilled concrete between the flanges. The moment resisting beam-column connections are field-bolted (with the heavy beam end-plates connected to the column flanges by means of high strength bolts). The composite beams form an integral composite slab-beam T-section with the concrete slab connected to the beam by means of welded headed shear

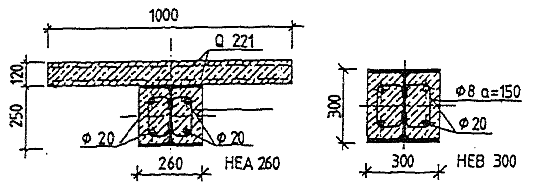

Fig. 3 Typical cross sections of columns and beams of composite frame

connectors or other connecting elements. The slab can either be poured in place or be formed by precast concrete elements. The poured slab can act as a composite slab when poured on top of a metal decking.

In order to minimize the shear stresses in the joint region, the horizontal stiffeners in the columns are placed at the same levels as the upper and lower edges of the beam end plates (Fig. 5). However, doubler plates are necessary in the joint regions to strengthen the web (lower the web shear stress).

5.1 PLASTIC DESIGN ASPECTS

A special problem in designing the composite frame has been the fact that the upper reinforcement of the T-beams in the slab can not be connected to the columns. However, care has to taken when applying the capacity design criterion since the tension force in the upper reinforcement may find a way to develop a resistance around the column through contact pressure between slab and column flange on the other side of the column. This would result in an increased beam-moment capacity which could lead to increased column web shear in the joint region and possible premature failure of the column joint region.

A very first linear elastic design analysis, in which a behavior factor of q = 6.0 had been assumed, lead to very high beam end moments due to combined earthquake and vertical loads. The design would become uneconomic and problems regarding the detailing of both the reinforcement and steel connections would arise. Hence, the following design-philosophy has been chosen:

The upper reinforcement for negative moments is reduced as much as possible. In fact the reinforcement has been limited to the minimum reinforcement required as stipulated by the EC 4 and EC 2 code provisions. This amount of reinforcement avoids uncontrolled cracking and associated plastification of rebars at large single cracks. At the same time, a check of crack widths under service loads was found to be satisfactory. Using this reinforcement layout, the design has been checked for the ultimate (limit) state considering the combination of static loads together with either fire or seismic loads. In this design check a linear frame analysis has been carried out in which the ultimate moments (obtained from the capacity analyses) have been introduced at the plastic hinge locations (i.e. critical regions according to EC 8 as actived for the given load direction). In these analyses the P-Delta effect has been included. Also the consequential increases of the bending moments in the mid-span region of the beams have been considered. In checking this earthquake design in general, increased values for the yield point values have been adopted.

Specifically, the nonlinear lateral load analysis performed has been based on the story shear forces resulting from a linear response spectrum analysis using q factor of 6.0. Based on these story shear forces, equivalent static loads (leading to the same story shear forces analyzed) were calculated and applied as external loads in the above mentioned frame analysis together with the assumed plastic hinges.

5.2 General Results

The results of the above described design analysis showed that the resulting positive beam-end moments were only half of the extreme negative moments. Thus it might be possible that the resistance of the beams may accomodate even higher earthquake loads than considered. Considering the relatively low positive end-moments, it should be noted, that the design of the beam-to-column connections for positive moments have been based only on these actual design moments and not on the plastic moment capacity of the adjacent beam cross section which is governed by the larger midspan moment under vertical loads. The columns have to be designed based on the ultimate capacity of the connecting beams. In the EC 8 chapter on composite strucures an overstrength-factor of 1.20 has been defined. However, similar to the stipulations of the EC 8 chapter on reinforced concrete frames, which permits the possibile reduction of the overstrength factor for concrete depending on a ratio of moments due to gravity and seismic loads, the overstrength factor of 1.20 for composite structures has been reduced accordingly.

As an alternative to considering the ill-defined increased moment resistance of the beam-end sections due to the upper beam-reinforcement, it might be preferable to prevent intentionally any contact between the floor slabs and the columns (e.g. by leaving a gap between slab and column filled with a soft material). This would avoid overstressing the beam columns joint as well

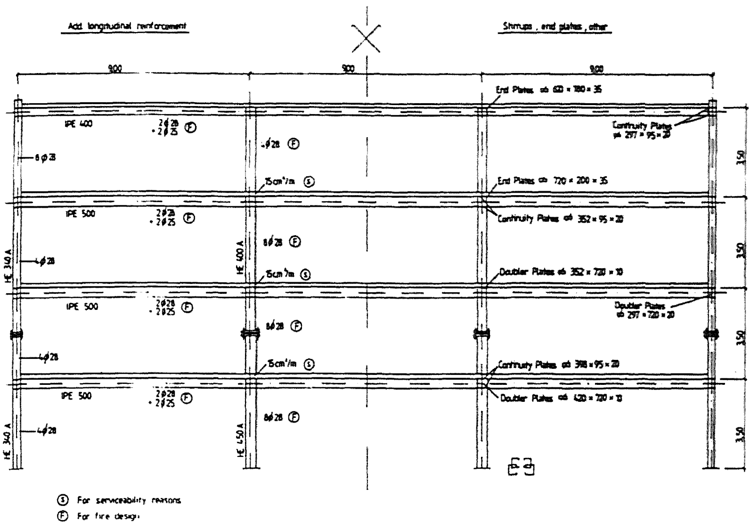

Fig. 4 Composite-frame design layout

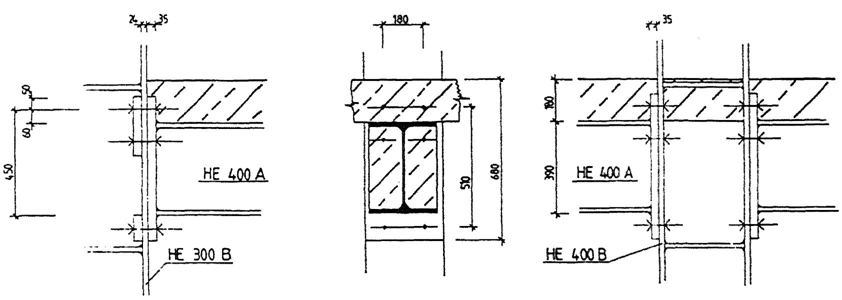

Fig. 5 Joint detailing in composite frame

as the column itself and thus reduce the strength demand for the columns in general.

In EC 8 specific provisions for confinement design in composite members are lacking. Hence, the relevant clauses given for reinforced concrete have been applied analogously for the composite frame design. However, the authors feel that further research in that area is necessary.

5.3 Relevant design checks

For the selection of the BEAM sections shear under the static ultimate limit state (no fire) governs. However, the design for fire-resistance requires additional lower longitudinal reinforcement in the beams of the exterior bays. For earthquake conditions premature failure in the critical regions e.g. due to insufficient confinement or due to failure of the shear studs should be avoided.

With respect to an economic layout and overall increased stiffness, IPE or welded sections instead of HEA sections could be used with advantage, also, since for larger IPE sections such as IPE 450 or larger offer a fully acceptable fire resistance.

The INTERIOR COLUMNS have to provide the main lateral stiffness in all cases. In fact the calculations showed that they are more critical under fire than under earthquake conditions. Also, the EXTERIOR COLUMNS are critical in case of fire exposure. Since the offer little of no contribution to the frame stability, they may be regarded as hinged columns and may even be so detailed.

The beam-column CONNECTIONS became critical for earthquake conditions since the utilization of the reinforcement in the concrete slab cannot be ensured.

Also, for the JOINT regions earthquake governs the design. Although the contribution of concrete in the shear panel zone has been taken into account, web doubler plates were found to be necessary.

It is interesting to observe how the stiffness of the beams influences the distribution of moments in the columns. For beams with low flexural stiffness, the moments in the columns of the upper stories tend to increase relatively fast. In fact, under these conditions the columns act more or less like columns hinged at one end with moments of identical sign over the full story height. Such moment distributions may lead to unacceptable plastic hinges in the columns; basically an uneconomical design which should be avoided. In order to provide a better fixity to the columns the use of IPE sections instead of HEA profiles for the beams is recommended.

Based on the studies reported herewith the necessity to adopt a plastic design approach became obvious. However, EC 8 currently does not give any directives in this matter. It is evident that a system designed for 'early' plastification will effectively be less stiff during the earthquake. This leads to reduction of the eigen-frequencies (approx. 30 % in the case presented) and in turn to a reduction of the spectral response. On the other hand the P-Delta effect becomes more pronounced. However, the potential increase of moments due to the P-Delta effect was offset by the decrease in the response due to the shift in the resonance frequencies.

6. CONCLUSIONS

The design examples showed that composite frames may be used as a viable alternative for conventional reinforced concrete frames also in earthquake prone regions. Prefabrication and reduced member sizes are clear advantages of the composite frame as compared to reinforced concrete frames. Moreover, this example study showed, that the composite frame - which was designed for a fire resistance class R 90 - was virtually automatically capable to withstand also severe earthquake loads. However, proper earthquake-resistant detailing of connections and critical regions is of paramount necessity.

REFERENCES

Ballio, G., A. Plumier & B. Thunus 1990. Influence of concrete behavior of composite connections, IABSE Symposium, Brussels.

Bouwkamp, J. & B. Schneider 1991. Seismic resistance of steel-composite-section (SCS) frames. Proc. 2nd Conference on Tall Buildings in Seismic Regions, Los Angeles.