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

Shear walls in hybrid construction for aseismic design

A.M. Pradhan & J.G. Bouwkamp
Institute of Steel Construction, Technical University, Darmstadt, Germany

ABSTRACT: A hybrid structural system with enhanced earthquake resistance, suitable for both RC ductile moment resistant frames and shear-wall buildings is presented. Enhanced resistance results from composite steel, concrete-filled, tubular column sections which provide, without stirrups, full confinement of typical longitudinal column reinforcement. Such columns can also be used as flange or edge-members of RC shear walls. Construction combines steel-erection of the prefabricated composite columns and RC construction of shear walls, beams and slabs; thus fully utilizing standard building practices. The paper addresses the structural design aspects of hybrid RC shear-wall buildings. Three basically different design solutions for the interface between the composite prefabricated columns, as edge elements, and the RC shear walls are discussed. Alternative solutions for each basic design are presented as part of a first-phase experimental test program underway at Darmstadt. Test specimen design and loading are also presented.

1 INTRODUCTION

Buildings designed as ductile moment resistant frames in steel or reinforced concrete perform well under earthquakes. Also, braced steel frames with beam shear links to dissipate energy in a ductile manner have found broad acceptance in aseismic design. Based on extensive studies by Ballio (1990) and Bouwkamp (1991) on steel-concrete composite beam-column connections and frames, such systems may find increasing use in earthquake resistant design.

In multi-story reinforced concrete buildings, flexural shear walls are often incorporated to provide a greater lateral stiffness than moment resistant frames. Basically, the ductile wall behavior can be assured through appropriate design of the reinforcing steel in both the walls (the web) and the vertical edge- or column elements (the flanges). However, often failure of the flange elements and walls in the lower regions occurs because of an inadequate layout of the reinforcement in these areas. Particularly, insufficient stirrup spacing in the flanges has led to serious earthquake failures. The same lack of confinement of the longitudinal reinforcing steel has caused also the failure of concrete frame columns under earthquake loads. Particularly, in countries where concrete is typically used for medium-rise buildings and both quality control and on-site inspection are poor, earthquakes have caused great devastation.

Failures could be prevented by a basic modification of the column and wall-flange design in both reinforced concrete frame and shear-wall buildings. A newly conceived structural system - called "hybrid" system - which is characterized by composite columns providing full confinement of the longitudinal reinforcing steel, is under development in Darmstadt for use in both moment-resistant ductile frames as well as in shear-wall buildings. In the following, only the hybrid system for structural walls is presented together with a review of the construction process, the different design solutions proposed for the structural implementation and a first- phase test program to assess the proposed design details. A sample design of a six-story hybrid building system with the composite shear walls is shown in Fig. 1.

2 HYBRID SYSTEM FOR SHEAR-WALL BUILDINGS

The special characteristic of the proposed hybrid system is the steel encased reinforced concrete composite column. While the columns or flanges, which are steel square or rectangular tubular sections filled with concrete, are erected in a typical steel construction manner, the remainder of the building, i.e. shear walls, beams and floor slabs, is formed and constructed like a typical reinforced concrete structure. Other then introducing the hollow steel column sections, the actual construction process fully draws on the experience of the building industry (concrete contractors) and thus does not require significant changes in the local or national construction practice. The steel tubular section in-herently provides continuous confinement of the longitudinal column reinforcement and concrete and allows eliminating all stirrups (except a few, necessary for assembling the reinforcement cage which is to be placed inside the hollow steel column section prior to concreting). In this manner, for little extra cost, failure due to poor design and/or inadequate placement of stirrups can be prevented automatically.

The hollow steel columns are to be fabricated in sections with nominal lengths of 1 1/2 stories for the first- and upper-floor columns and 2 stories for the intermediate columns. The lower column sections will be provided with typical base plates and longitudinal extended column reinforcement for later anchoring. Alternatively, also an extended

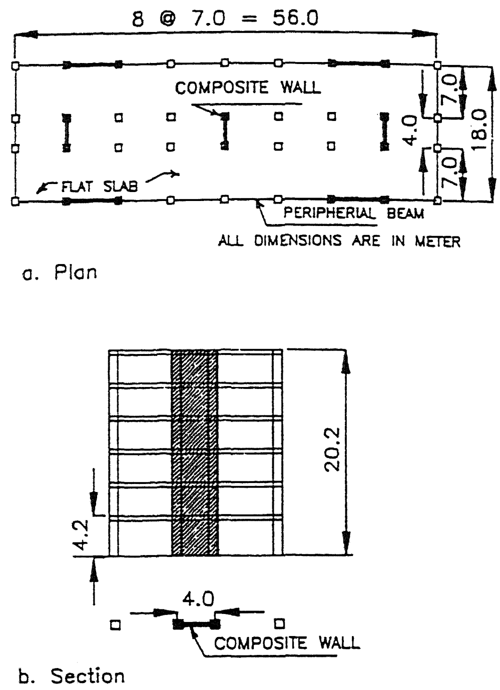

Figure 1. Hybrid (structural wall) building system

column length to permit placement and anchorage in a foundation hole is possible. The other ends of the column sections are designed typically for later field erection. Construction of the composite columns most likely will be done by the concrete contractor either in a prefabrication yard or on site. In this process, first, the steel reinforcement cage will be placed inside the hollow column section. Secondly, the reinforcing steel to connect shear wall column section will be stuck inside the section through holes drilled in the tube wall. Also, with the floorbeams being designed as collector beams and/or as beams of a ductile moment resistant frame, additional reinforcing bars will be placed through the hollow steel section to provide later continuity with the typical beam reinforcement. Finally, the concrete will be poured to fill the column section. Design alternatives for the interface between column and shear wall will be presented in the next section.

In the typical construction process, the prefabricated composite column sections with extended interface reinforcement will be erected by the steel fabricator. Subsequently, the concrete contractor places the reinforcing steel for the shear walls, which is designed to withstand the earthquake design forces (capacity design) both in the wall and near the column-wall interface. After building the formwork for the walls, floor beams and slab, the reinforcing steel for the beams and floorslab will be placed. In this process, the beam reinforcement has to be designed to provide full continuity with the reinforcing steel extending from the prefabricated steel composite tubular section. Finally, the concrete will be poured. The building construction will be continued by systematically following the construction sequence described above.

3 DESIGN CONSIDERATIONS AND DETAILS

The basic cross-section of a composite wall is shown in Fig. 2.

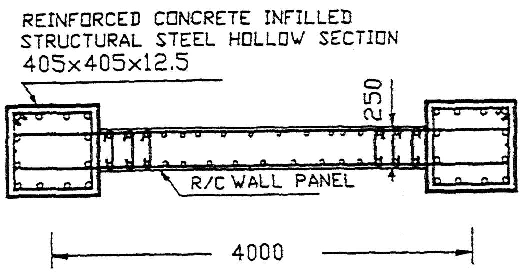

Figure 2. Composite structural wall

The tubular sections are standard CIDECT sections with overall dimensions which may very in 2-inch intervals between 8 and 16 in. square. The wall thicknesses may very between 0.25 and 0.375 in. for 8 x 8 in. sections and 0.375 and 0.50 in. for 16 x 16 in. sections. For square sections with overall dimensions of 10, 12 and 14 in. the maximum available wall thickness is 0.625 in. The steels available have specified yield stresses of Fy = 36 and 52 ksi (reflecting steels specified as Fe 250 and 360, respectively). The above material selection provides a considerable range for an effective design, considering both the confining effect of the wall thickness as well as the axial capacity of the overall cross-section. Considering construction cost-efficiency, particularly in developing countries, the wall thicknesses of the tubular sections should be based on the confinement requirements and kept to a minimum. In certain countries the final composite column design may not only depend on the design-load requirements but also on the availability and quality of the reinforcing steel and concrete. Also, in case usable-space requirements are important smaller steel sections with larger wall thicknesses and even higher strength concrete may be selected effectively.

For hybrid-wall systems, the design of the interface between the shear wall and the column edge members is critical. A number of different designs for the interface connections have been developed and are presently studied at Darmstadt as part of a first-phase experimental program. In the design of the interface connections two different design philosophies can be pursued. In one case, considering the desirable ductile behavior for earthquake resistant design, the aim would be to develop an interface design which would provide a ductile behavior immediately at or near the interface. However, considering the inherent brittle behavior of concrete and the possible rapid cyclic deterioration which may occur near the interface, a second design approach may be preferable. In that instance, the interface should basically be designed with sufficient overstrength to prevent serious cyclic deterioration in the column-wall interface and the immediately adjacent region (up to about a distance of at least half the wall thickness from the actual interface). At the same time, through a reduction of the reinforcing steel in an area away from the interface, the design would be aimed at developing ductility in that region. In the following sub-sections three basic designs are discussed.

3.1 *Interface connection with reinforcing bars*

Basically, this interface design uses typical reinforcing bars which are stuck through holes in the steel tubular column wall facing the wall. These bars which are typically placed in pairs may be straight or having hooks inside the column to provide sufficient anchorage (see Fig. 3a). The bar-size and location must at least correspond with the horizontal reinforcing steel of the wall. However, a closer spacing (e.g. half the spacing distance of the wall reinforcement) with the same bar sizes (see Fig. 3b) may not only improve the immediate interface load

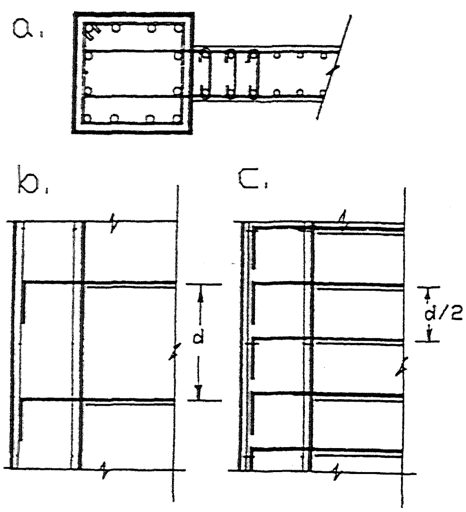

Figure 3. Interface connection with horizontal reinforcing bars

transfer but also strengthen the direct interface region (design philosophy 2). Although interface dowel action of the reinforcing bars would likely be more critical for small diameter bars, a smaller bar-diameter arrangement with an even closer spacing (e.g. a quarter or a third of the spacing distance between the horizontal wall reinforcement) may well offer an improved cyclic response. In general transverse wall ties in the interface region will enhance the concrete confinement and consequently the force- transfer resistance in the interface region.

Rather than developing an interface design with horizontal bars, a better solution may well be attainable by using an array of diagonally (45 degree) arranged reinforcing bars (see Fig. 4a). Although in this figure the bars are shown without hooks experimental studies will be performed to assess the possible benifits of hooked bars in this application. This diagonal bar arrangement would transfer the interface forces together with the concrete in a truss-like manner and thus constitutes a distinct improvement over the dowel-like transfer (and possible splitting effect) typical for a horizontal bar design. In comparison, the bar sizes in the diagonal bar design can be reduced because of the tension-compression rather than the bar-shear transfer. In this design, the holes in the column wall can be arranged in sets of 4 with a vertical spacing distance equal to the spacing distance of the horizontal wall reinforcement (see Fig. 4a) or in sets of 2 spaced at half this distance (see Fig. 4b). Of

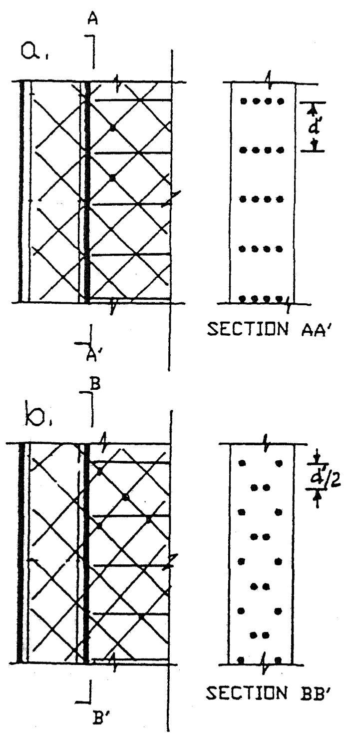

Figure 4. Interface connection with diagonal reinforcing bars

course a certain offset between holes has to be maintained to prevent interference between the individual bars and between those bars and the column and wall reinforcement. A comparison of the arrangements given in Figs 4a and 4b shows that the more denser array of case 4b tends to reflect the design philosophy 2. A one-third vertical spacing distance between the pairs of holes may further improve the cyclic behavior. Also in this type of solution transverse wall ties will improve the regional resistance and load-transfer behavior.

3.2 *Interface connection with headed shear studs*

In this case the load-transfer connection is designed using basically 3/4 in. headed shear studs welded to the tubular column wall (see Fig. 5a). Although this solution is simple to fabricate the main problem potentially lies in the danger of developing a vertical planar splitting of the reinforced concrete shear wall. Hence, cross ties in the interface region between the two wall- reinforcing meshes or mats are absolutely necessary. In relation herewith the relative position of the studs vertically with respect to the main shear-wall mesh reinforcement, as shown in Fig. 5b, may have an effect on the shear transferring behavior. Because a reduction of the local load-transfer forces between wall and studs would undoubtedly improve the overall shear load transfer the shear-stud type of connection may be improved by replacing the single-row of 3/4 in. diameter studs with a double row of 1/2 in. diameter studs. In general, a reduction of the stud diameter size as well as the vertical stud-spacing, thereby creating a denser load-transferring stud arrangement, may provide structurally a better solution. A further possible improvement of the load-transfer behavior (reducing the danger ofpossible splitting) may result by introducing a certain off-set between alternating studs (see Fig. 5c). Also, in case of two rows of studs, the load-transfer might be improved by changing alternatingly the horizontal distance between two adjacent studs.

A general effect which will undoubtedly influence the interface load-transfer behavior can be attributed to the wall thickness of the composite tubular column member. This parameter influences the effective fixity of the welded studs under increasing deterioration and the subsequent shear transfer stiffness. In order to study this effect in the Darmstadt tests, column sections with different wall thickneses will be investigated. Also the behavior of a thin-walled section with a thicker bar welded over the full length of the column will be investigated.

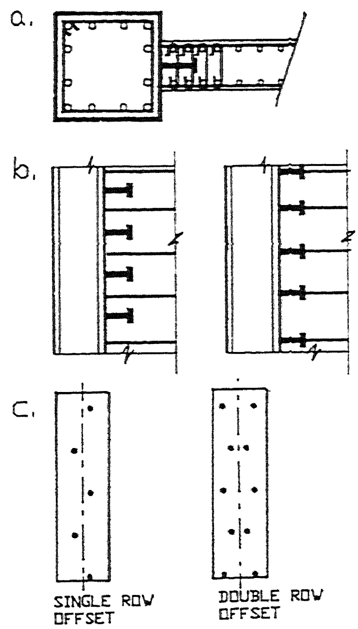

Figure 5. Interface connection with headed studs (19mm dia.)

3.3 *Interface connection with perfobond strip*

A possible third connection design involves the use of so-called perfobond strips as shown in Fig. 6. These steel strips, which are welded continously to the tubular column section, are 60 mm (2-3/16 in) high and 12 mm (0.50 in.) thick and are

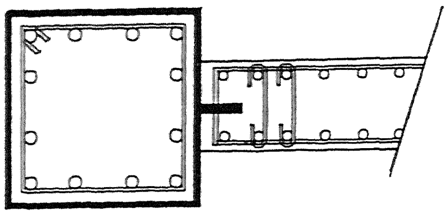

Figure 6. Interface connection with perfobond strips

perforated with 35 mm (1-3/16 in.) holes spaced at 50 mm (2 in.). Fatigue studies by Andrae (1990) of composite girders whereby these strips were used to connect the reinforced concrete deck flange to the flange of the steel girder showed excellent results. Hence, because of the structural simplicity of this design, an assessment of these strips at the interface connection of wall and tubular edge section seems of interest. While the earlier tests were performed without transverse reinforcement placed through the holes of the strips the Darmstadt test program will study this arrangement. As an alternative, the test program at Darmstadt shall also study the behavior of two parallel strips under shear-load transfer. The transverse reinforcing bars, placed through the holes of the perfobond strip, will complement the normal transverse ties interconnecting the two wall-reinforcing meshes or mats. This arrangement is necessary to prevent an early planar splitting of the concrete shear wall.

4. TEST PROGRAM

In order to study the different effects of the three basic interface design solutions a first-phase test program is presently underway at Darmstadt. In order to make a first assessment of the different design alternatives discussed above the load transfer between columns and wall is studied under monotonic fluctuating loads. A typical test setup as shown in Fig. 7 reflects the connection design for the building shown in Fig. 1 under a medium size earthquake (UBC Zone 3).

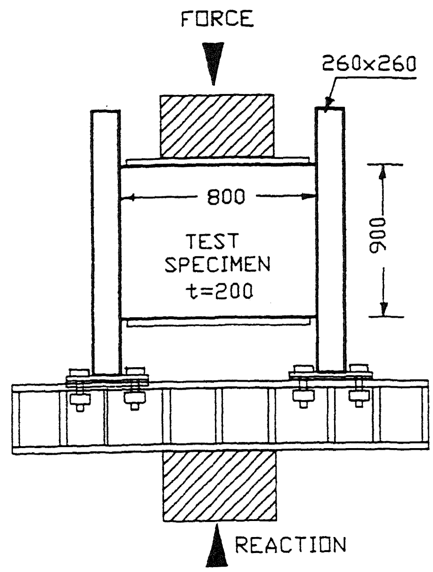

Figure 7. Test setup

5. CONCLUSION

Conceptual design studies indicate the feasibility to improve the aseismic design of RC frame and shear-wall buildings by replacing the RC columns and shear-wall flange elements by steel composite (concrete filled) tubular column sections. The different connection details presented for a composite-RC shear-wall building system, called HYBRID System, allow the development of a ductile response under earthquake loads. Results of a first-phase test program will provide data for further design improvement and system development.

REFERENCES

Andrae, H.P. 1990. Economical shear connectors with high fatigue strength, *IABSE Symposium*, Brussels.

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.