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HYBRID SHEARWALL SYSTEM — SHEAR STRENGTH AT THE INTERFACE CONNECTION

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Abstract. Based on a series of alternating, displacement-controlled load tests on ten one-third scale models, to study the behaviour of the interface of a hybrid shear wall system, it was proved that the concept of hybrid construction in earthquake prone regions is feasible. The hybrid shear-wall system consists of typical reinforced concrete shear walls with composite edge members or flanges. Ten different anchorage bar arrangements were developed and tested to evaluate the column-shearwall interface behaviour under cyclic shear forces acting along the interface between column and wall panel. Finite element models of the test specimens were developed that were capable of capturing the integrated concrete and reinforcing steel behaviour in the wall panels. Special models were developed to capture the interface behaviour between the edge columns and the shear wall. A comparison between the experimental results and the numerical results shows excellent agreement, and clearly supports the validity of the model developed for predicting the non-linear response of the hybrid wall system under various load conditions.

Keywords: aseismic design, earthquake, hybrid structural system, reinforced concrete, shearwall.

1. Introduction

Although current codes cover well defined rules for the aseismic design of reinforced concrete buildings, earthquakes continue to cause extensive structural damage, particularly in developing countries. In most cases, the significant damage is triggered by column failure in the lower stories of RC frames. In addition, the failure of reinforced concrete shear walls has been found to be caused by local failure of the edge-member columns (flanges). In most instances, column failures of this kind are triggered by an inadequate layout of the stirrups in the column end-regions. The consequent lack of confinement of the concrete core, and associated buckling of the longitudinal reinforcing steel, lead to the observed failures. Considering these observations, a new hybrid structural system for both moment resistant frame and shearwall buildings has been proposed by Bouwkamp (1992) and studied at the Darmstadt University of Technology.

Basically, this system is characterized by replacing the typical reinforced concrete columns or shearwall edge members with concrete-filled rectangular steel tubes acting as composite columns. Conceptually, the remainder of the structural system is planned to be identical to the layout of a typical reinforced concrete building. Basically, the tubular section provides direct confinement of the concrete core and serves as a longitudinal reinforcement. In fact, depending on the thickness of the column wall, additional typical column reinforcement may not be necessary. Of course, it is also possible to minimize the tube-wall thickness by only satisfying the confinement requirements, and to design the longitudinal reinforcement as for a normal, non-composite, column. However, the major objective in developing the proposed hybrid structural system is to design a connecting interface between

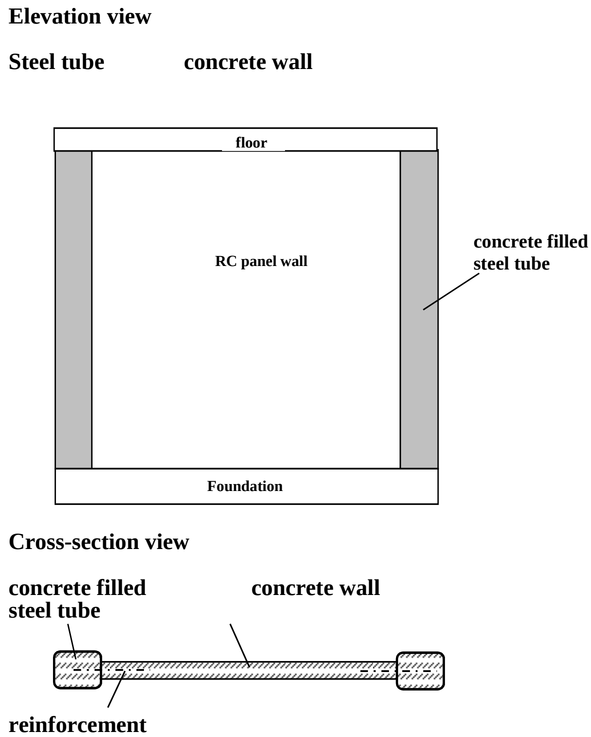

Figure 1. View of HSW.

the composite-columns and the reinforced concrete elements. Optimization of the composite column design therefore does not form the subject of our study. Instead, extensive experimental and numerical studies of the seismic behavior of the proposed hybrid system, as affected by the reinforcement in the interface regions, have been performed in Darmstadt.

A research program on the design and the seismic response of hybrid moment resistant frames was carried out successfully by Ashadi (1994, 1997). The results showed that this system can be used effectively in ductile moment resistant frames under earthquake exposure. Parallel studies have focused on the potential use of this system for shearwall type buildings. In this case, too, the main effort has focused on the design of the reinforcement at the interface between the composite column, or flange, and the concrete shear wall. Various design solutions have been studied experimentally under cyclic shear loads acting along the column-wall interface. Experimental findings and recommendations for the effective and economical design of the interface connection (IFC) of a hybrid shearwall (HSW) for use in regions of high-seismicity is presented here.

Considering the construction process of the specific hybrid system, the composite columns are to be prefabricated at either a special construction yard or on site. Before pouring the concrete in the hollow column sections, the interface reinforcement has to be installed through predrilled holes in the tube column walls. This reinforcement requires on one end hooks, for anchorage into the column concrete core, and on the other end a sufficient length, for connection (lapped bars) to either the common beam or slab reinforcement, in case of moment resistant frames, or to the typical shearwall reinforcement. The composite concrete filled rectangular steel tubes (CFRST) are erected in a typical steel construction manner. The design of the footing of the first-floor columns, which extend over 1.5 or 2.5 stories, can follow typical steel design procedures. Because the possible additional longitudinal column reinforcement can extend from the bottom of the column, alternative connection designs between column and foundation are possible. Subsequent columns are prefabricated in 2-story long sections, and are erected following typical steel construction procedures.

2. Experimental program

Several alternative interface designs have been developed and tested in working on an optimum design solution for the interface connection between the RC wall panel and the composite edge member. For this purpose, a 6-bay by 4-bay, 8-story high hybrid shearwall building with plan dimensions of 36 m by 20 m and 28 m in height was designed according to the provisions of EC-8, zone 4; the design of the shear wall reinforcement was based on EC-2. The resulting design also in fact satisfies UBC 1991, Zone 3 and the

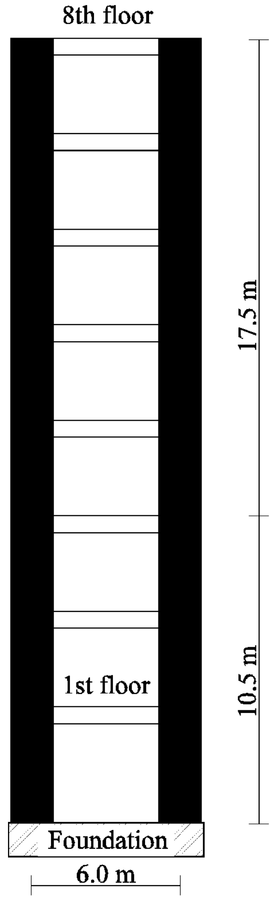

FIGURE 3. Elevation view of HSW.

US code ACI-318-89 code requirements. The structural layout, as shown in Figure 2, calls for four shear walls oriented parallel to each of the two main axes. The elevation of a 6 m wide 8-story high shear wall is shown in Figure 3. An equivalent static analysis, based on the typical code-specified force distribution, was used in the design of the prototype structure.

Because of cost considerations and test capacity limitations, a reduction in the scale of the shear wall system was necessary. As the response of the model in studying alternative RC wall panel-to-edge member interface design solutions should be as representative as possible of the cyclic response of the actual shearwall, a reduction in scale to not more than one-third of the actual wall was considered acceptable. This meant that it was possible to use common readily available reinforcing bars, which would properly model the tensile force resistance of the reinforcement. However, because of space and test-capacity limitations, it was not possible to test an entire one-third scale shearwall model, or even a 3-story high portion of it (see Fig. 4). Hence, with the aim of studying alternative interface connection designs between column and RC wall panel, only the edge portion of the first-story shear wall was selected for our study.

A perspective view of a typical test specimen is shown in Figure 5; because of the laboratory test conditions, it was necessary to mount the actuator

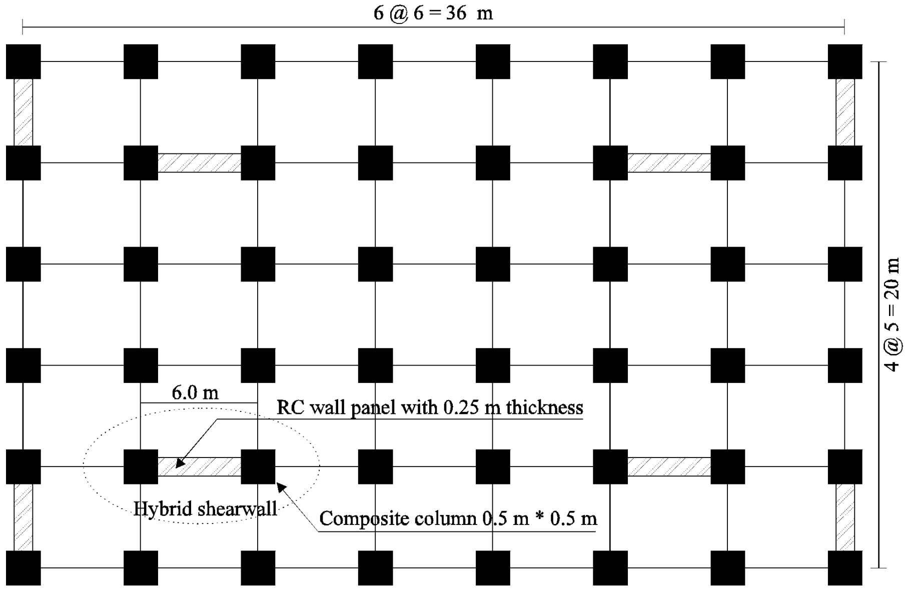

Figure 2. Plan view of building.

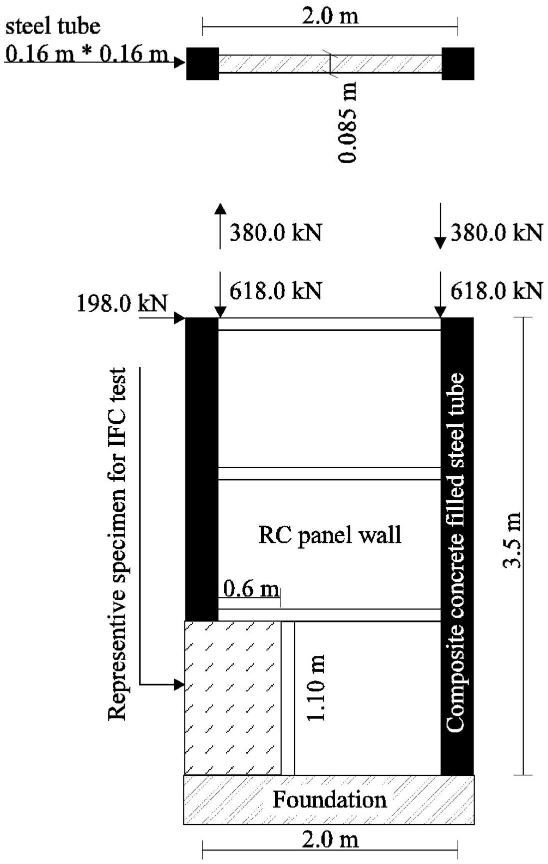

Figure 4. 1/3 scale of hybrid shearwall (HSW).

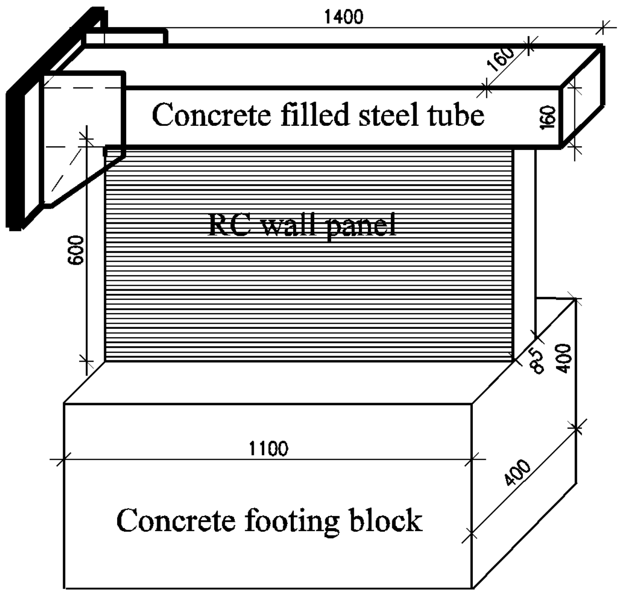

Figure 5. General view of test specimen.

horizontally. Hence, the composite edge member — with outside dimensions of 160 mm by 160 mm and a tube-wall thickness of 5.6 mm — had to be placed in horizontal position. The wall of the test specimen, 110 cm in length and 68 cm in height (being a portion of the shearwall width, measured to the edge-member centre line), represents about one-third of the first-story model shearwall. The wall panel thickness had been set to 8.5 cm (or basically 1/3 of the prototype wall thickness of 25 cm). In order to anchor the test specimen to the test frame, the wall was cast integrally with a concrete footing block with overall dimensions of 40 × 40 × 110 cm. Because of cost restrictions, it was decided not to model the floors as edge elements on either side of the test specimen, but rather to introduce additional reinforcement along the free edges of the wall in order to counter the overturning moment introduced by the horizontal cyclic interface load. This was considered acceptable because of the basic shear loading of the wall specimen.

An overall view of the test setup showing the test specimen, the double-acting actuator and the test frame is presented in Figure 6. The test specimen was typically anchored with HS bolts to the upper

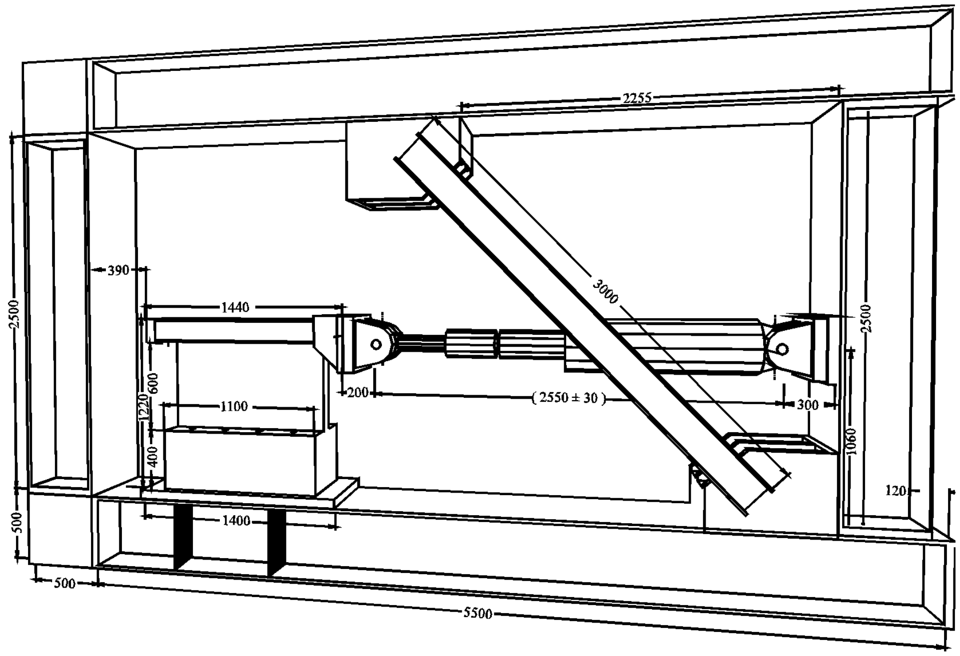

Figure 6. Test setup.

flange of the lower beam of the test frame. Under horizontal displacement-controlled cyclic loading, a certain rocking of the test specimen could occur due to deformations of the upper flange of the lower steel beam of the test frame. Vertical web stiffeners were therefore added to stiffen the upper flange directly below the test specimen. The position of the actuator was dictated by the need for the forcing level to coincide with the level of the interface between edge-member and shearwall.

2.1. Test specimen design

The design of the model shear wall called for a double-layered 10 × 10 cm mesh layout with ⌀ 6 mm bars vertically and horizontally. Because the test specimen reflects a squat shear wall and premature test failure of the wall (prior to the interface connection), the horizontal bars in the test specimen were increased from ⌀ 6 mm to ⌀ 8 mm. The vertical bars were kept as in the model shear wall, namely, ⌀ 6 mm at 10 cm spacing. Conceptually, an interface layout of ⌀ 6 mm anchor bars spaced at 10 cm, to match the ⌀ 6 mm bar arrangement in the wall panel, would in fact be a dowel shear transfer, and would not be adequate to develop an appropriate interface force transfer, as the concrete would be inactive at this junction. It was therefore decided to develop an interface design with increased bar diameters, namely, two layers of ⌀ 8 mm, 30 cm long, anchor bars spaced at 10 cm and lapped with the corresponding ⌀ 6 mm bars of the RC wall panel reinforcement.

An alternative design that we decided to test had a primary two-layered arrangement of ⌀ 6 mm, 30 cm long anchor bars spaced at 10 cm, and lapped with the ⌀ 6 mm bar reinforcement of the RC wall panel, plus an additional array of 30 cm long ⌀ 6 mm anchor bars, placed mid-way between the 10 cm spaced primary bars. This basically 5 cm spacing arrangement of ⌀ 6 mm anchor bars is shown in Figure 7a. In the case of the first test specimen, the ⌀ 8 mm anchor bars have the same layout as the primary ⌀ 6 mm bars shown in Figure 7a. The advantage of these two designs is the relative ease with which the hooked anchor bars can be placed into the hollow steel column section and held in position, prior to placing the concrete in the prefabricated columns. In addition, it is relatively easy to tie the basic shear wall reinforcement to the 30 cm long anchor bars, which reflects standard practice.

A third design, conceptually similar to the first two, was developed with a primary arrangement of two layers of 30 cm long ⌀ 6 mm anchor bars spaced at 10 cm. However, in order to increase the dowel shear transfer capacity, additionally, single welded — 12 cm long — shear studs 3/8 in. in diameter (9.52 mm) were placed midway between the ⌀ 6 mm anchor bars, thus resulting also in a spacing distance of 10 cm. From a construction point of view, this design has the same advantages as the first two designs. However, a (potential) disadvantage is the need for the steel fabricator to be licensed to weld typical headed shear studs. An additional consideration in selecting such a design is the local bending resistance of the column wall to provide the necessary bending resistance at the base of the shear studs.

Considering that the state of shear in the wall leads to the development of an inclined set of normal forces, it was decided also to test two layouts with 45-degree inclined double layered anchor bars. Therefore, a 4th design was formed with two layers of ⌀ 8 mm anchor bars spaced at 10 cm. These bars had 45-degree hooks both on the inside of the column and at the extending end of these bars. The hooks at the extending end

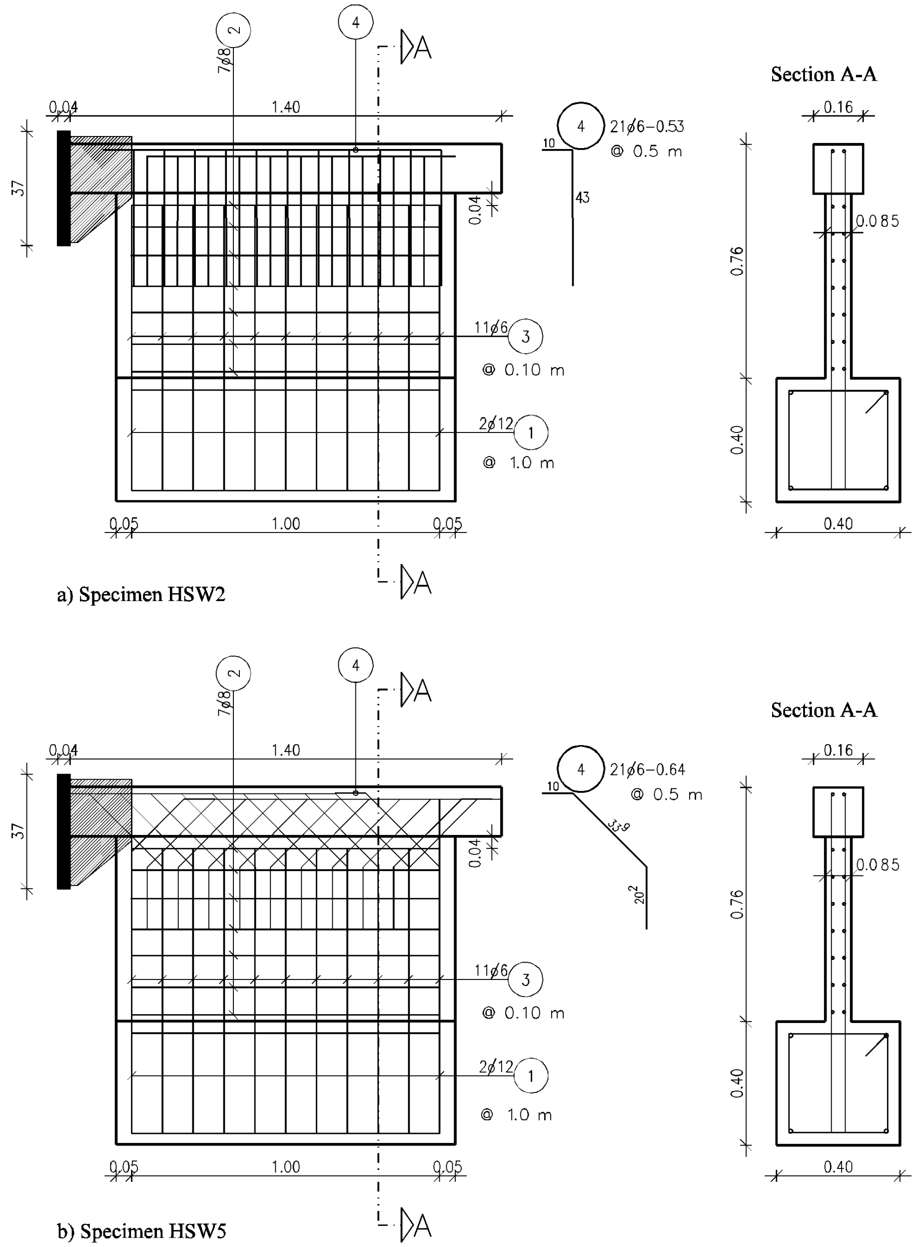

Figure 7. Reinforcement details at the interface connection.

were introduced to allow the horizontal ∅ 6 mm bars of the RC wall panel to be lapped to these anchor bars (the kink of these hooked anchor bars coincides with a horizontal ∅ 8 mm bar). Similar to the fourth design, a fifth design (see Fig. 7b) was conceived as having a primary array of ∅ 6 mm, 45-degree, anchor bars spaced at 10 cm. This design further called for a second two-layered set of ∅ 6 mm, 45-degree, anchor bars to be placed at the midpoint of distance between the primary bars thus resulting in an overall spacing distance of 5 cm.

A final — sixth — design to be tested had an anchorage arrangement of open ∅ 8 mm stirrups welded to the wall of the steel column tube (no wall penetration) at a spacing of 10 cm. These stirrups were lapped with the two-layered ∅ 6 mm bars of the shear wall reinforcement.

A summary of the six different test specimens covering the various design solutions for the interface connection (IFC) is given in Table 1.

In the test specimens, the composite steel tube was of grade Fe 360 steel. The concrete had been specified as C30 and the deformed reinforcing bars as BSt 500. Unfortunately, the concrete quality varied considerably; for the different specimens, the material test values reflected concrete qualities of C46, 46, 22, 33, 29 and 33, respectively.

2.2. Instrumentation and Test Sequence

With the aim of studying the column-wall interface connection under a displacement-controlled cyclic load acting along the interface and applied to an end-plate arrangement on one end of the composite column, the instrumentation was designed to evaluate both the

Test specimen	Test panel reinforcement		
number	horizontal	vertical	Interface connection
1	⌀ 8 mm	⌀ 6 mm	Straight anchor bars (⌀ 8 mm) at 10 cm
2	⌀ 8 mm	⌀ 6 mm	Straight anchor bars (⌀ 6 mm) at 5 cm
3	⌀ 8 mm	⌀ 6 mm	Straight anchor bars (⌀ 6 mm) at 10 cm plus (⌀ 9.52 mm) with 12 cm long shear studs at 10 cm
4	⌀ 8 mm	⌀ 6 mm	Diagonal anchor bars (⌀ 8 mm) at 10 cm
5	⌀ 8 mm	⌀ 6 mm	Diagonal anchor bars (⌀ 6 mm) at 5 cm
6	⌀ 8 mm	⌀ 6 mm	Open stirrups with straight anchor bars (⌀ 8 mm) at 10 cm welded to the steel tube wall

Table 1. Hybrid Shear Wall System — test specimens.

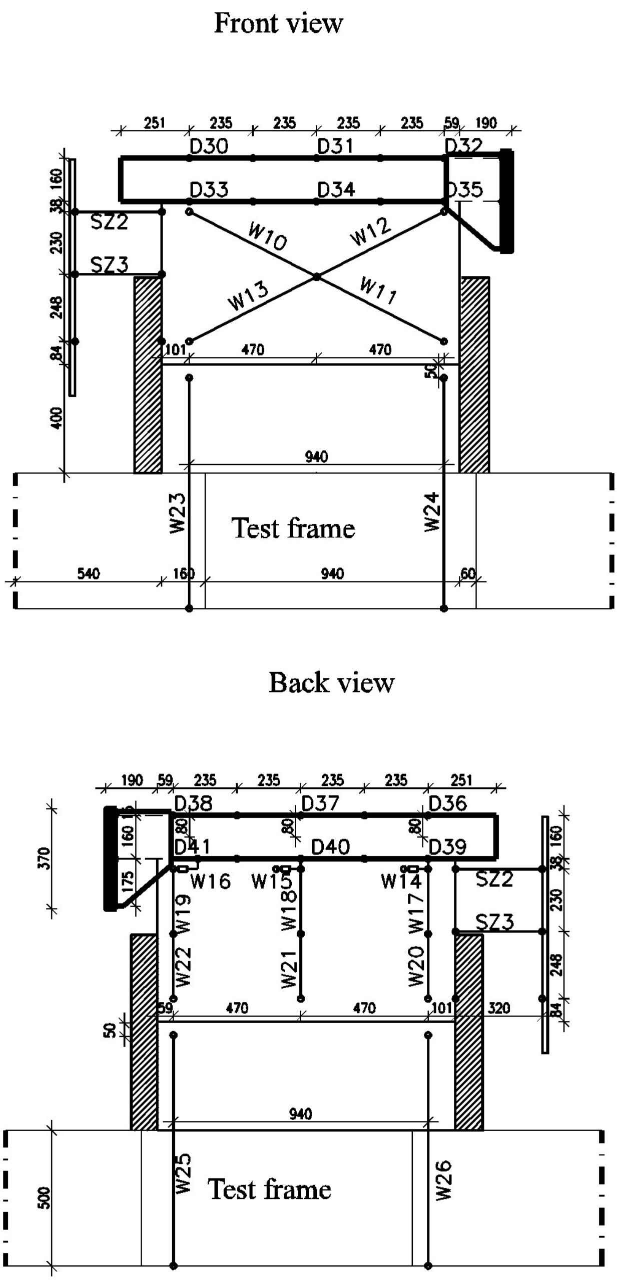

Figure 8. Instrumentation of a test specimen.

shear wall and the interface behavior. Basically, the different interface connections were designed with the intention that the resistance of the connection would be larger than the actual wall panel capacity. In order to evaluate the behavior of the different parts of the test specimen, the layout of the instrumentation included both displacement transducers and straingages (see Fig. 8).

Other than the typical test control transducers for the displacement control and loadcell output, LVDT displacement transducers W10 through W22 were used to record the interface slip (W14–16), and the shear panel deformations (W10–13, diagonally and W17–22, vertically). W23–26 displacement potentiometers were used to measure possible base rocking motions.

In addition, strain gages (D30–41) were placed in three sections along the length of the steel column tube; these measurements were intended to provide information about the load transfer along the length of the interface connection. Relative displacements (slip) along the column-shearwall interface were to be recorded by displacement transducers W14, 15 and 16 (with a gage length of 7 cm). In addition, displacement potentiometers (SZ 2 and 3) were used to measure the overall shear distortions of the shear wall.

During the preliminary test phase, a number of test specimens were tested with a larger number of displacement transducers on the shear wall. In these tests, the edges of the shear wall were totally free (as shown in Fig. 5). The results showed that the bending effect in the shear wall initiated a failure of the wall immediately above the anchorage beam before any distress in the interface region could be observed and a rating of the different interface connections could be made. We therefore decided to reduce the bending moment effect in the shear wall by introducing additional side support to the wall over the lower half of the test specimen (see Fig. 8). This decision also resulted in a reduction in the number of transducers used in the final tests (however, for general data reduction and comparison the transducer numbering

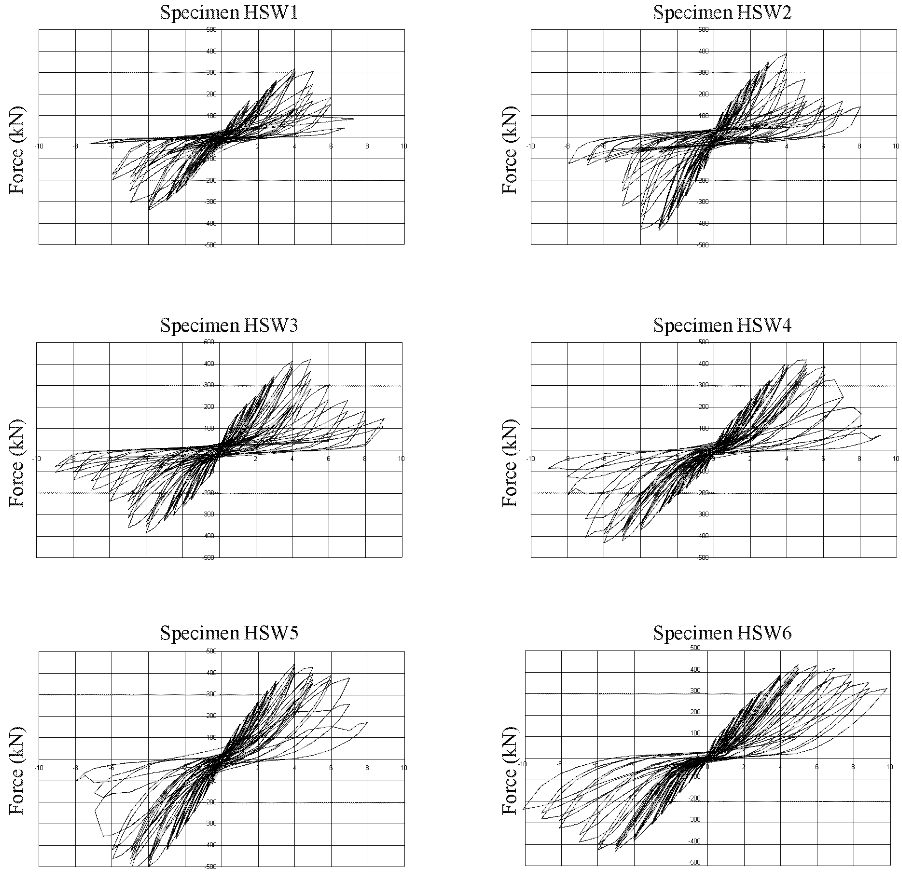

Figure 9. Force — displacement diagrams.

system was kept the same).

The tests were performed under displacement-controlled conditions (measured against the motion of the free end of the tubular column. The alternating displacements were increased in 0.5 mm intervals from ± 0.5 to 3.0 mm and in 1.0 mm intervals from ± 4.0 to ± 7.0, 8.0, 9.0 or 10.0 mm, depending on the performance of the specific test specimen. At each displacement step up to ± 4 mm, the specimen was subjected to three cycles of loading. Subsequently, in order to assess the deteriorating behavior under repeated displacement at ± 5 mm, four cycles of displacement were introduced. Finally, from ± 6 mm on, each displacement step was introduced twice. The maximum displacements to which the test specimens were subjected were governed by the observed performance.

3. Test results

In general, it can be noted that the hysteretic response of all specimens exhibited the pinched force-displacement response common to cracked shear loaded concrete specimens. The first three specimens, with straight anchor bars and, in the case of Specimen 3, welded shear studs, failed at the interface connection. In the other cases, both Specimens 4 and 5, with 45-degree inclined diagonal anchor bars, and Specimens 6, with welded straight open stirrups, failed in the shear wall due to an excessive concrete contact pressure at the edges.

3.1. Force — displacement

The hysteretic force displacement data for all six specimens are presented in Figure 9. Other than Specimens 1 and 2, which registered maximum resistances of about 320 kN and 390 kN, respectively, the remaining specimens developed maximum resistant values between about 420 and 450 kN.

Considering the deterioration under repeated alternating cyclic displacements, the first three specimens with straight anchorage bars, as compared to the other three specimens, show a distinct loss of resistance. Specimen 1, which had shown little loss of resistance up to a 3-cycle alternating displacement of ± 4 mm, exhibited after 3-cycles at ± 5 mm a drop in resistance of 30 % (after 4.5 cycles the loss in resistance had increased to 50 %). The same basic phenomenon was observed for Specimen 2. In this case, after a virtually no-loss observation under a 3-cycle alternating displacement of ± 3 mm, a loss of 30 % was observed after 3 cycles at ± 4 mm (increasing to 40 % in the next half cycle). In addition, Specimen 3 showed a similar behavior as Specimen 1. In this case, the loss of resistance under a 3-cycle displacement of ± 4 mm was still minimal. However, at ± 5 mm, a loss of resistance of about 25 % after 3 cycles increased to 40 % after 4.5 cycles.

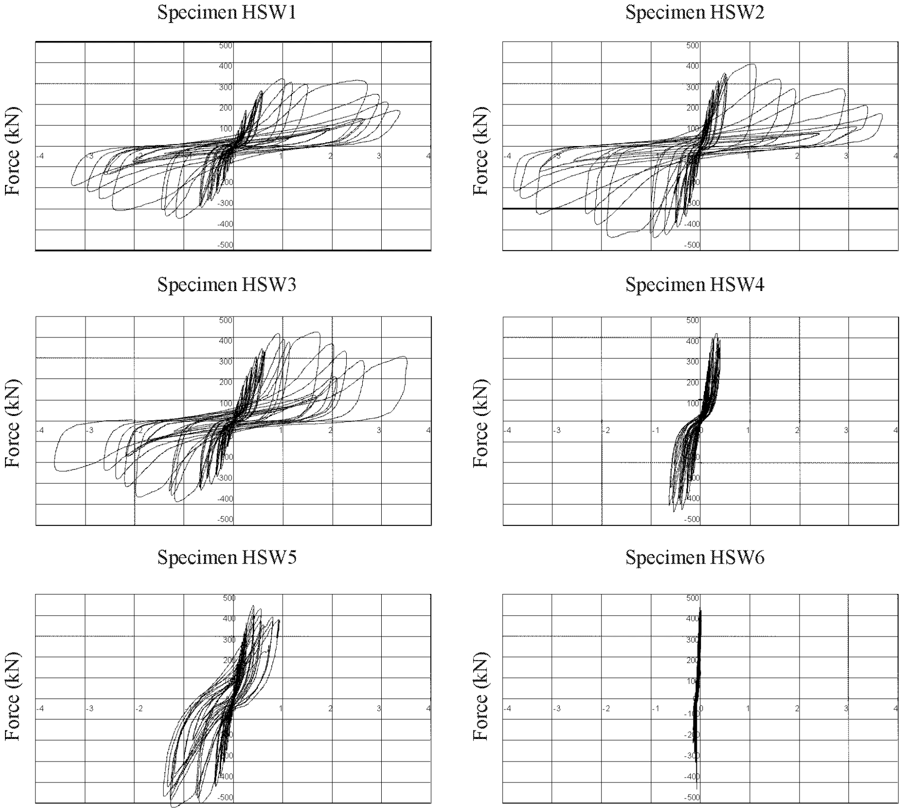

Figure 10. Force — slip diagrams.

In comparison, the three other specimens exhibited both a relatively gradual drop in resistance under increased displacements and relatively little deterioration under repeated alternating cyclic deformations. Comparing the results for Specimen 4 and 5, Specimen 5 with a 5 cm interval of ⌀ 6 mm diagonal anchorage bars, shows a slightly better response than Specimen 4, with 10 cm spaced ⌀ 8 mm anchorage bars. Specimen 6, with ⌀ 8 mm stirrups spaced at 10 cm, shows an almost identical response to that of Specimen 4 up to an alternating displacement of ± 4 mm (with a similar ⌀ 8 mm anchorage arrangement at an interval of 10 cm). However, under increasing cyclic displacements Specimen 6 shows a superior response as compared to both Specimen 4 and 5. In fact, at a cyclic displacement of ± 10 mm, the drop in cyclic resistance of Specimen 6 was only 25 %. On the other hand, at a cyclic displacement of ± 9 mm Specimens 4 and 5 were no longer able to resist a significant interface shear force.

3.2. Force — slip

For all six specimens tested here, the force applied to the test specimen versus the slip of the edge member column relative to the shear wall, measured along the interface at the middle of the column (LVDT W15 — see Figure 8), is presented in Figure 10. The slip, measured against a 7 cm gage length, reaches displacements of close to ± 3.5 mm for the first three specimens, with straight anchor bars and welded shear studs (in Specimen 3 only). For the specimens with diagonal anchorage bars, the slip reaches maximum values of between 0.5 and 1.5 mm. For the welded straight stirrup anchorage arrangement, the slip is virtually negligible.

4. Conclusions

A hybrid system consisting of typical reinforced concrete wall panels with composite edge members or flanges has been studied experimentally. The edge members, which are formed by composite hollow steel square column sections, are prefabricated with reinforcing bars anchored inside the column and extending through the wall of the column for connection to the wall panel reinforcement. The remainder of the building is constructed like a typical reinforced concrete structure. Based on a series of alternating, displacement-controlled load tests on six one-third scale models, to study the behavior of the interface connections between column and wall panel, it can be concluded that hybrid shear wall construction in earthquake prone regions is feasible

The six interface connections, which were subjected to cyclic alternating displacement-controlled shear forces, had basically two different types of interface designs. Three test specimens had straight (horizontal) column anchorage bars passed through holes in the tube wall, and two had instead diagonally oriented bars extending through the tube wall. A sixth specimen was basically similar to the first three, but had horizontal anchorage bars welded to the wall of

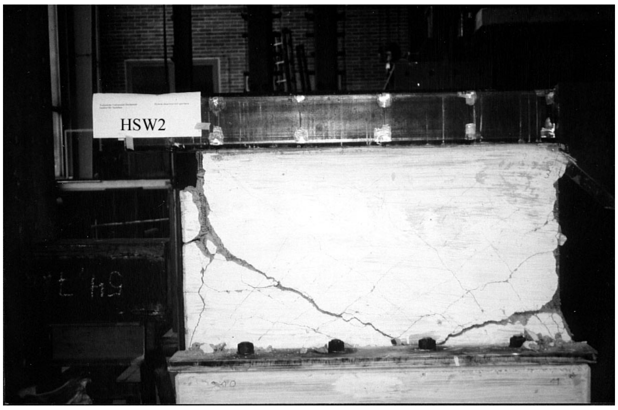

Figure 11. HSW2 after test.

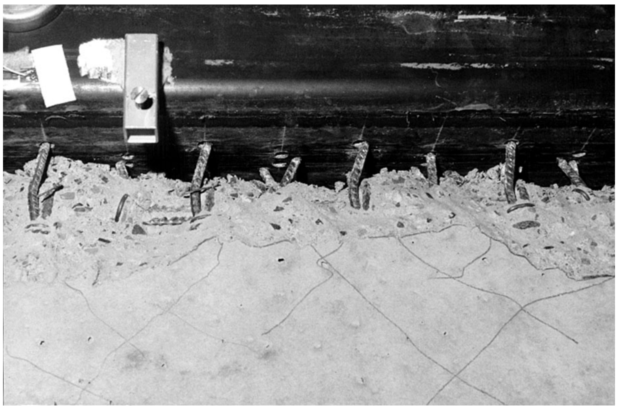

Figure 13. Detail HSW7 after test.

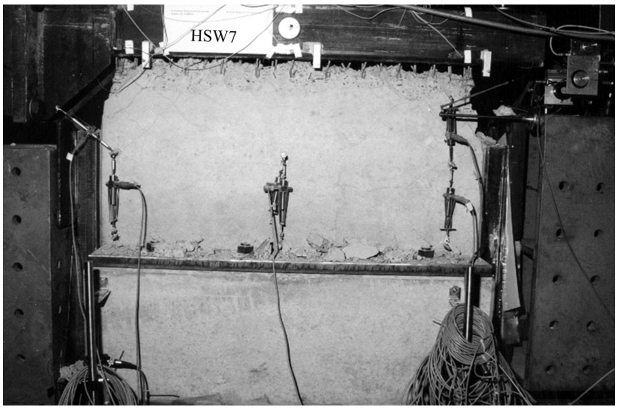

Figure 12. HSW7 after test.

the tubular column. The results showed that the diagonally arranged bars at the column-wall interface performed better under cyclically induced alternating interface shear-loads than the interface connections with horizontal anchorage bars. However, the alternative design with horizontal anchorage bars welded to the column wall showed the least slip between edge member and wall.

In turn, such a design would exhibit the lowest level of energy dissipation at the edge-member and wall interface. To develop the integrated behavior of the hybrid shear wall, at this stage of the study we recommend either diagonally arranged anchor bars extending from the column, or welded horizontal stirrups to be connected (lapped) to the shearwall reinforcement.

In order to study the overall seismic behavior and the load carrying capacity of the hybrid shear wall, it is recommended to test three 1/3-scale hybrid shear walls subjected to cyclic alternating, displacement-controlled shear forces. As the critical shearwall region is assumed to be the first three stories of the building, the tests will be carried out on test structures three stories in height. However, as cost considerations and test limitations do not permit an experimental study on full scale models, it is proposed to study the seismic behavior of three 1/3-scale models of the three-story high hybrid shearwall (Fig. 14) with three different interface connections.

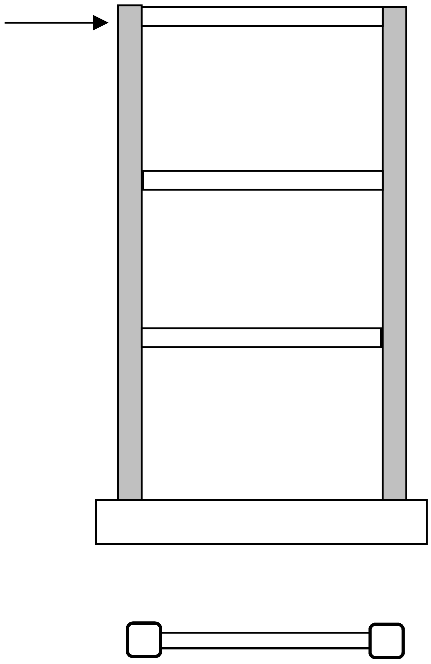

Figure 14. View of the test wall.

Acknowledgements

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