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Large-Scale Testing of Steel Concrete (SC) Composite Connections between Walls, Slabs and Foundations

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ABSTRACT

Steel Concrete (SC) composite structures are an appealing alternative in industrial sectors currently dominated by reinforced concrete (e.g. nuclear power plant, foundations to offshore wind towers, bridges and marine structures). In many cases it may lead to a faster construction and cost saving. Actually the design of SC structures in Europe is impeded because it is not covered by normative regulations in the Eurocode. In order to improve the understanding of the mechanical behaviour and to develop and implement regulations into the Eurocode design concept a 4-year research program sponsored by the European Research Fund for Coal & Steel has been initiated in 2013. One of the project's tasks was to investigate the behaviour of wall to wall, wall to slab and wall to foundation connections. This task has been performed in collaboration of the Materials Testing and Research Institute (MPA Karlsruhe), Karlsruher Institute of Technology (KIT) and the engineering company SMP Ingenieure im Bauwesen GmbH. This project has been completed in 2017.

In a first step numerical investigations have been performed in order to better understand the complex mechanical interrelation and flow of forces between steel plates, concrete, tie bars and studs in the connection region. In this step an optimization of possible connection configurations has been performed in terms of strength and ductility. This work has again shown that the modelling of SC elements – in particular the interaction between studs and concrete – is crucial and needs specific considerations. Based on these findings different connection types have been specified and experimentally tested up to failure. The failure mechanisms predicted in the numerical investigations were in good agreement to experiments. The findings from the investigations are presented for the wall to wall connections.

INTRODUCTION

A steel-concrete-steel composite (SC) structure is constructed with two steel plates that are connected with tie bars and serve as permanent formwork. Studs are welded on the inner surface of the steel plates that tie the concrete and steel plates together and transfer shear between them. Then concrete is placed in between. SC can lead to faster construction and cost saving in industrial sectors in comparison to reinforced concrete. The advantages are mainly the elimination of formwork, the elimination of reinforcing bars, the ability to simple post-connect of equipment by welding it anywhere to the steel plates and the transfer of a considerable amount of work from site to fabrication shops. While in US (AISC N690), Korea (KEPIC-SNG) and Japan (JEAC 4618) specific regulations exist in Europe this type of construction is impeded because the design is not yet covered by the Eurocode regulations.

Due to the lack of regulations in Europe a 4-year research program sponsored by the European Research Fund for Coal and Steel (RFCS) has been initiated in 2013 and ended in early 2017. In the framework of the project necessary data on the behaviour of SC structures at ambient and elevated temperature had to be produced using a combination of design studies, advanced numerical analysis and a series of large size tests. The aim was to develop a design guide for SC structures conform to the Eurocode design concept. The design guide was demonstrated on a reference building example. Further information can be found in Burgan (2015) and the final report that is submitted.

The project was subdivided into 9 working packages. The authors where mainly involved in the working package No. 4 (WP4) 'Testing of SC connections'. The main objective of WP4 was to study the behaviour of SC connections and to generate new experimental data on the resistance and ductility due to different types of loading conditions (bending, shear). Connection configurations can vary significantly and it was not possible to examine all possible configurations. Therefore WP4 was focused on three representative connection types: wall to wall, floor to wall and wall to foundation connections. The force transfer mechanisms between elements and the associated failure modes were studied. To achieve this, a series of large scale connection tests were performed.

This paper gives an overview on the experimental tests performed, the interpretation to the experimental test results and the numerical simulation with the selected type of modelling for the wall to wall connections. The numerical simulations accompanied the tests from planning phase to interpretation of results. The development of design rules were part of working packages No 7 to 9 and is not covered by this paper. Furthermore the floor to wall and wall to foundation connections are not part of this paper.

INVESTIGATED CONNECTION

A T-shaped wall to wall connection was investigated by experimental tests and numerical simulations. Two different stud configurations inside the connection region were considered:

- The type I connection had one row of studs inside the overlapping part of the two walls (Figure 1 left). This type was tested in bending dominated loading only (Figure 2, left).
- The type II connection had three rows of studs inside the overlapping part of the two walls (Figure 1 right). This type was tested in bending and shear dominated loading (Figure 2, right).

The steel plates inside the overlapping part were perforated with Ø133 mm holes in order to ease concreting on building sites. Due to that weakening the thickness of these plates inside the connection region was increased from 8 mm to 12 mm.

Each connection specimen was equipped with different kinds of measuring devices. Strain measurement gauges were applied to the steel plates. Displacements were measured with inductive displacement sensors and draw wire sensors. A special optical measurement system was used to measure deformations of the liner surface at the backside of each connection.

Table 1: Material of components with properties from material testing.

component	material	properties from material testing
concrete	C30/37 - CEM III/A 32.5 N	$f_{c,cyl}$ = 34.5 to 44.6 N/mm², $E_{c,s}$ = 25.9 to 30.8 kN/mm²
steel plates	steel grade S355J2+N	f_y = 371 to 408 N/mm², f_u = 542 to 543 N/mm²
studs	steel grade S235J2+C450	$f_{p0.2}$ = 454 to 499 N/mm², f_u = 486 to 543 N/mm²
tie bars	steel grade S355JR+AR	$f_{p0.2}$ = 441 to 451 N/mm², f_u = 566 to 570 N/mm²

The selected material of each component is shown in table 1. The properties of the steel plates and tie bars were determined in accordance with EN 6892 using reference samples made of the same material. For the studs the values were taken from the material certificates of the manufacturer. The concrete compressive strength was determined using stored concrete cubes and cylinders that were tested at the day of each connection test. The relevant mechanical properties of concrete were carried out in accordance with DIN EN 12390 (compressive strength of cubes $f_{c,cube}$, compressive strength of cylinders $f_{c,cyl}$, tensile splitting strength of cylinders f_{ct}) and DIN EN 1048 (secant modulus of elasticity in compression of cylinders $E_{c,s}$). The range of test results is given in table 1 for each material. A major difficulty was to reach a specified concrete strength because concrete manufacturers in general assure only a minimum strength. So the strength may be larger. Although this point was discussed with the manufacturer the strength still varies significantly.

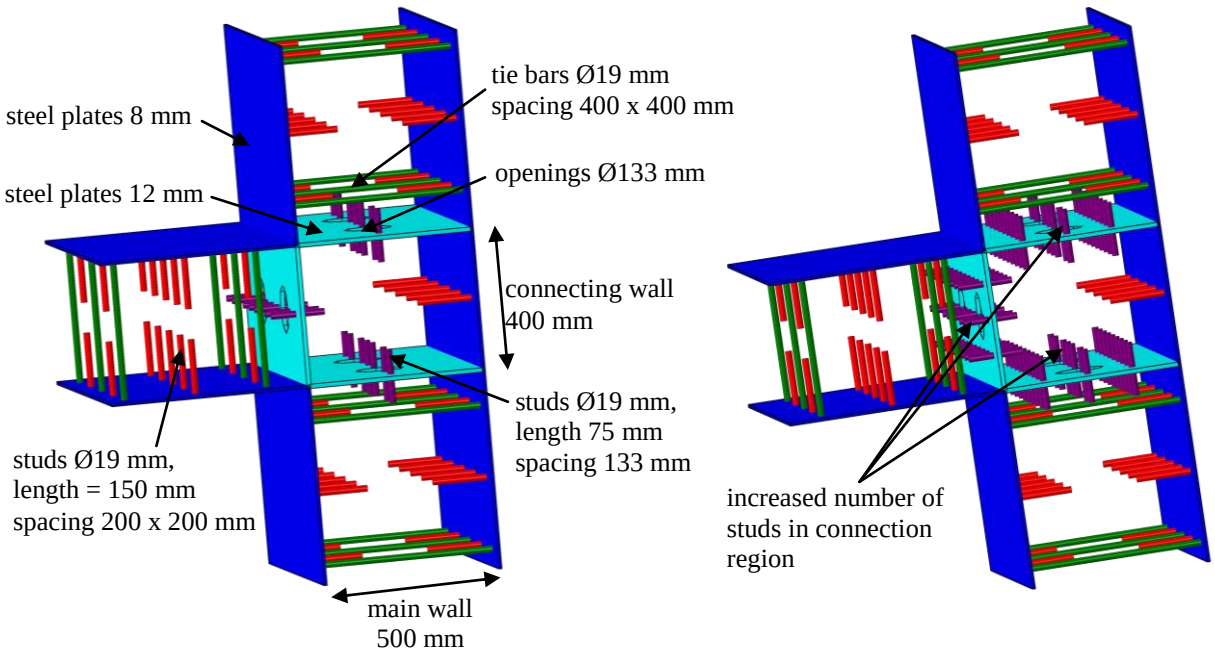

Figure 1. Wall to wall connection type I (left) and type II with increased number of studs (right)

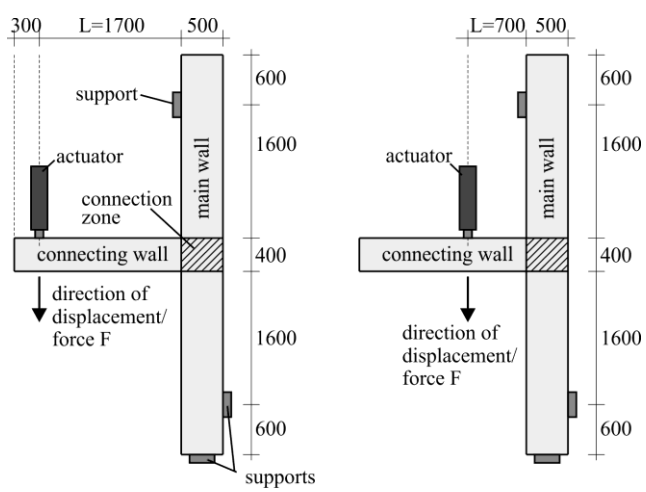

Figure 2. Wall to wall bending dominated (left) and shear dominated (right) test setup

EXPERIMENTAL TEST RESULTS

All tests have been done at the Materials Testing and Research Institute (MPA Karlsruhe). The testing speed was quasi static with less than 1 mm/min. Figure 3 shows the specimen wall to wall type I in bending loading at the end of the test. The nominal (characteristic) flexural resistance acc. to the US code AISC N690s1-15 in the connecting wall is M_R = 1022 kNm. This value is in good agreement to the Korean and Japanese codes. With the lever arm L this corresponds to a force at the actuator of

$$F = M_R / L = 1022 / 1.70 = 601 \text{ kN} \quad (1)$$

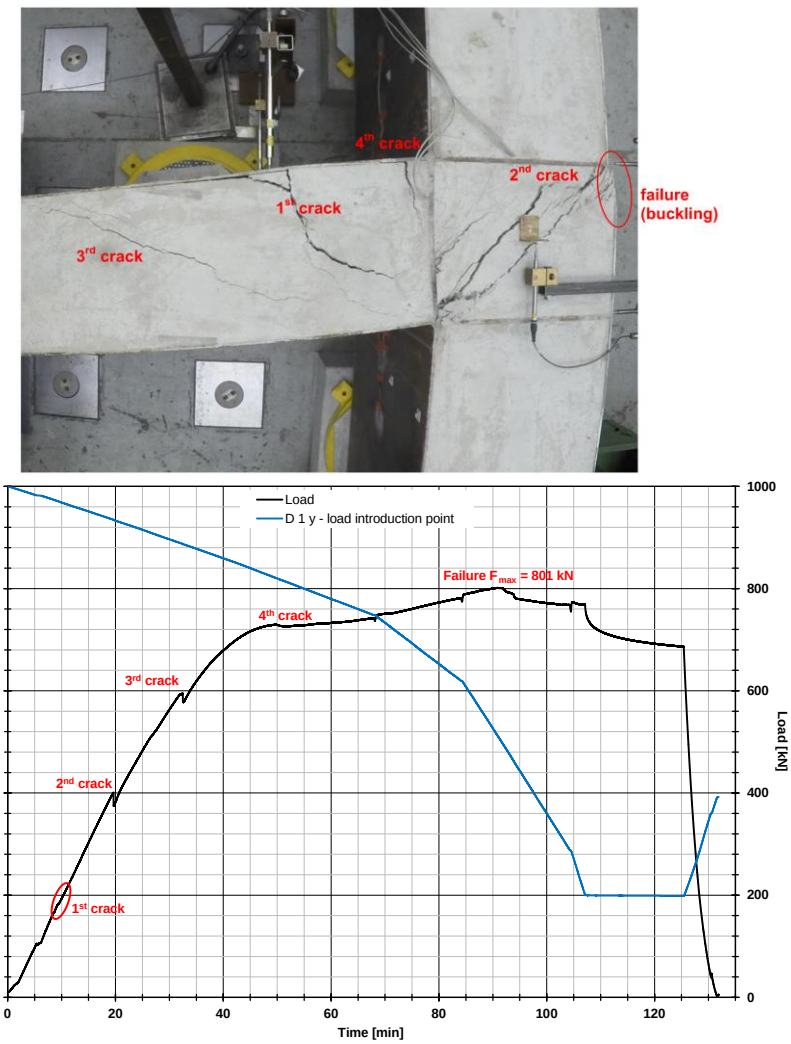

Figure 3. Wall to wall type I in bending loading at the end of the test (top) and loading curve (bottom)

The first crack occurred at the connecting wall in shear at 200 kN. This was somewhat surprising because the shear resistance predicted by the codes is far beyond this value (cp. next chapter). However, this first crack did not reduce the structural stiffness as can be seen in figure 3. Then an uncritical diagonal crack occurred along the compression strut inside the connection when the moment in the connecting wall had reached 680 kNm (66% of nominal resistance). This crack successively increased. However the design load could well be reached. Then the moment further increased up to 1362 kNm until it slowly decreased. This process was accompanied by intense cracking inside the connection, spalling of concrete, a

successive buckling of the back-side liner plate and pronounced shear cracking in the connecting wall. The test was stopped at a displacement of 200 mm. Converted to the centre line of the connection a maximum moment of 1562 kNm (corresponds to 152% of nominal flexural strength of the connecting wall) was reached. In conclusion it can be said that the connection was stable in terms of moment-rotation resistance however with limited cracking, concrete spalling and buckling of the back-side liner plate. It is probable that the early shear failure in the connecting wall was due to the interaction with bending.

Figure 4 shows the specimen type II in bending loading at the end of the test. The first crack occurred like at the type I specimen at the connecting wall in shear at 200 kN. However the crack in the connection occurred at a later stage – at 850 kNm (83% of nominal resistance). This crack stayed very fine and no concrete spalled out of the connection. In this test the nominal flexural resistance, which was the same as in the type I test, could again well be reached. However, the whole test was accompanied by intense shear cracking in the connecting wall and only a maximum moment of 1282 kNm was reached because of a brittle shear failure of the connecting wall. Although the maximum moment is smaller than at the type I test the connection performed better because at this stage the connection was stable without concrete spalling, buckling of the back-side liner plate and only a fine crack. From the author's view it can be assumed that the connection is not the reason for the reduced shear resistance of the connecting wall at type II test. This rather comes from the unavoidable scatter of concrete strength in the connecting wall.

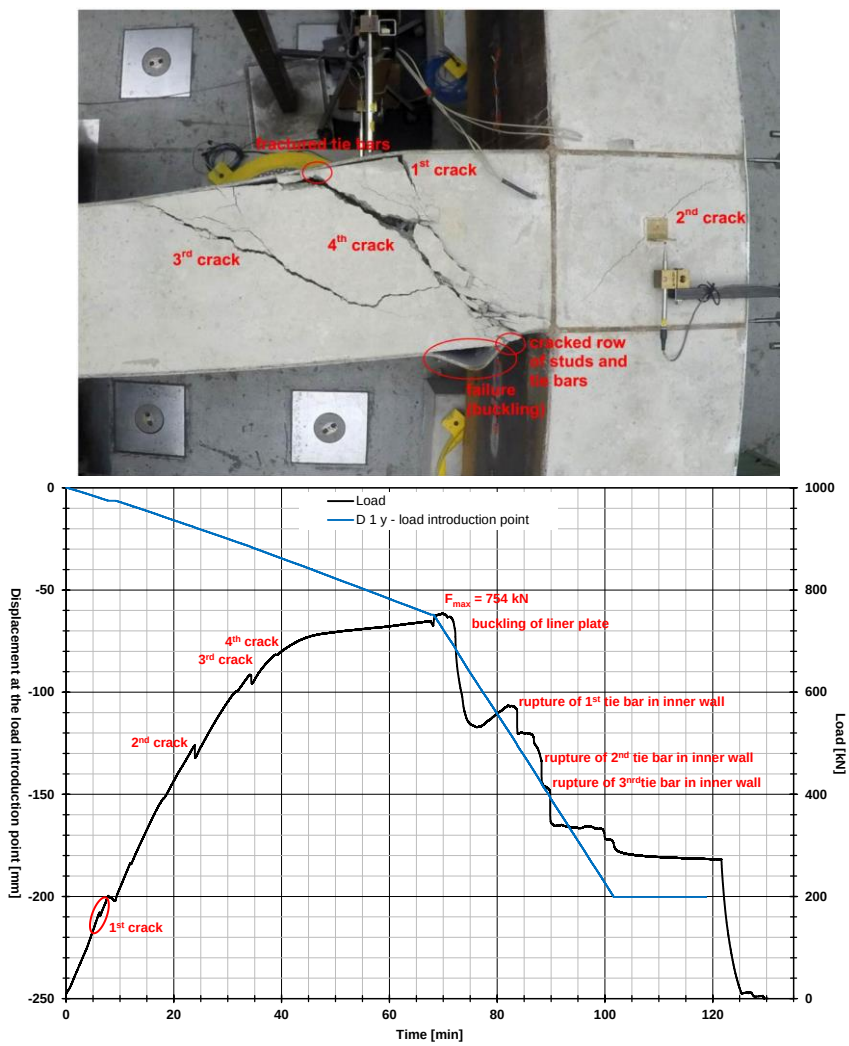

Figure 4. Wall to wall type II in bending loading at the end of the test (top) and loading curve (bottom)

Figure 5 shows the specimen wall to wall type II in shear loading at the end of the test. Because shear strength predictions highly diverge in the different codes it was calculated acc. to three different codes: The US-code AISC N690-12, the Korean code KEPIC-SNG and the Japanese Code JEAC 4618. While the US-code results in a low resistance of only 308 kN the Korean code agrees well with 1164 kN to the Japanese code with 1289 kN. The early cracks in shear at all wall to wall tests support the US equation which results in the lowest resistance. However the wall section always had enough robustness to well reach the resistance acc. to the Korean and Japanese code. Inside the connection an uncritical diagonal crack occurred along the compression strut when the shear force reached 1120 kN. This crack stayed fine until the maximum shear force of 1617 kN was reached. Then the resistance rapidly decreased due to a shear failure in the connecting wall. With increasing deformations, welds between tie bars and steel plates failed. In conclusion it can be said that the connection was stable up to complete failure of the connecting wall.

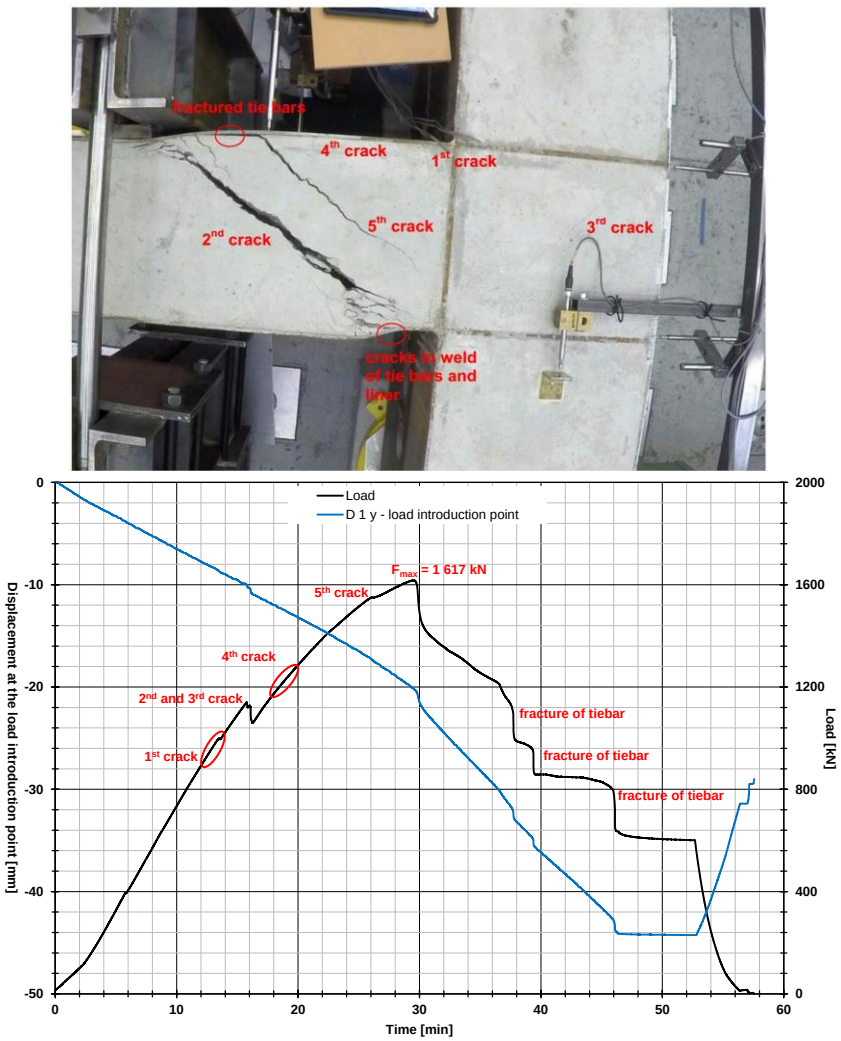

Figure 5. Wall to wall type II in shear loading at the end of the test (top) and loading curve (bottom)

NUMERICAL INVESTIGATIONS

In order to better understand the behaviour of SC connections at high utilisation ratios numerical investigations have been done in advance of the tests and during interpretation of test results using the

Finite Element program ABAQUS/Explicit. The explicit solver was used because complex contact problems can be solved more straightforward in ABAQUS. In literature many variants of modelling approaches can be found. They reach from economic ones with multilayer composite elements where the total section is considered by one element e.g. Hrynyk et al. (2015) to complex ones where all components are modelled explicitly e.g. Varma et al. (2011), Kurt et al. (2015). In the present task the latter approach was chosen in order to capture the complex flow of forces in the connections.

Two types of models with different degrees of detailing have been developed (Figure 6). In both models concrete is mapped by continuum elements and steel plates by shell elements. However at the simplified model concrete is regularly meshed independently of the tie bars and studs. The tie bar/stud elements are superposed on the concrete mesh and are rigidly connected with concrete at their total length (constraint: 'embedded'). In the detailed model openings are provided in the concrete mesh for the tie bars/studs. Interaction between concrete and tie bars/studs is realised by contact surfaces with friction (μ = 0.4). A rigid connection is only realised at the stud heads. In order to better capture the concentrated yielding of tie bars/studs at the fixing to the steel plates continuum elements have been used in the detailed model while beam elements were used in the simplified model.

At both modelling variants the same constitutive material laws have been used: For concrete the '*concrete damaged plasticity*' model was selected that considers the nonlinear stress strain relation in compression (crushing) and in tension (cracking) both for arbitrary spatial states of loading. For steel the '*Johnson Cook*' model was selected that considers yielding and hardening.

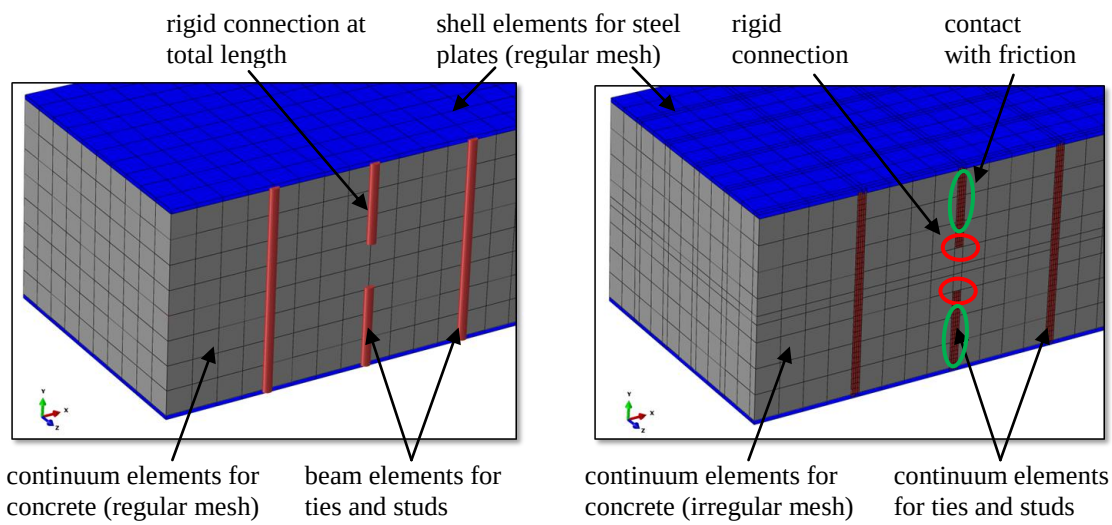

Figure 6. Simplified model (left) and detailed model (right)

Figure 7 shows the load displacement curves of the wall to wall connections from experimental tests in comparison to the numerical simulations. The tests and simulations were conducted up to complete failure of the specimen. An exact agreement between tests and simulations was reached up to 25 to 40% of ultimate load at the bending tests. Then the models underestimated the stiffness with the higher deviation at the simplified model. At the type I connection the ultimate load of 0.8 MN was underestimated by ~12% at both modelling variants. At the type II connection the resistance in the simulation was 0.76 MN at the detailed model and agrees well with the test result. At the simplified model the force incorrectly still increases at high displacements. The behaviour inside the connection with a diagonal crack with

higher strains in the type I connection compared to the type II connection could be predicted with the detailed model. However the correct failure mechanism in the connecting wall was not predicted. While the simulation predicts a bending flexural failure in the test a shear failure occurred. This inadequacy leads to different displacements at ultimate load. Furthermore it is the reason for the discrepancy of the softening branch in the post ultimate load regime at the type II connection.

At the shear test the results of the simulation and the test almost agree up to 50% of the ultimate load. The small deviation comes from uncorrected support settlements in the test. While the ultimate load was underestimated by 12.5% in the simulation the displacement at ultimate load was exactly predicted. In this case the shear failure in the connecting wall could be predicted by the simulation, too.

In conclusion it can be said, that the simulations with the detailed model well predict the mechanical behaviour in the connection with a diagonal cracking compared to the test. However somewhat surprising is that the flexural resistance in the connecting wall is underestimated in the simulations leading to a flexural failure instead of a shear failure compared to the tests and that the early cracking in shear is also not predicted.

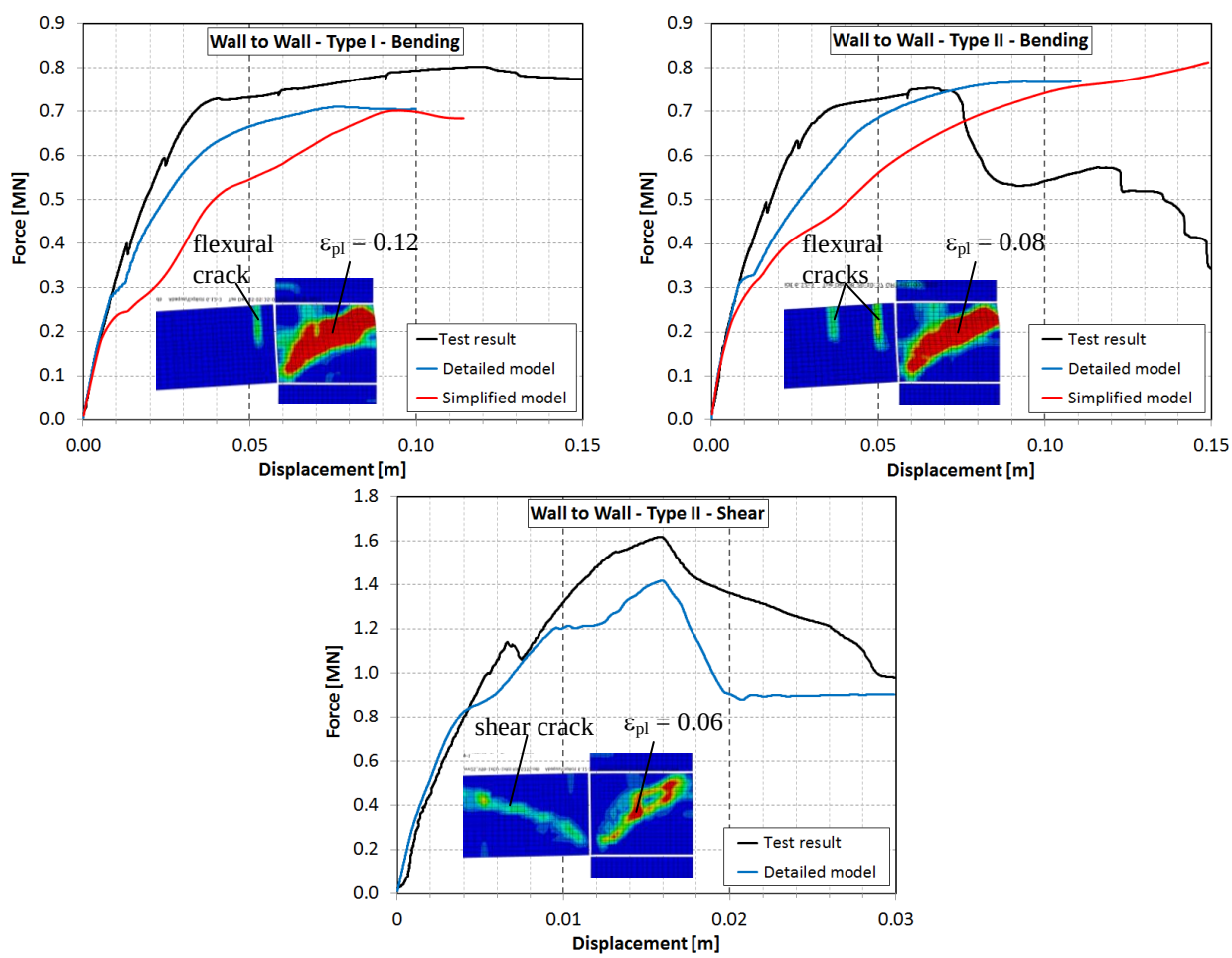

Figure 7. Comparison of test results to numerical results from simplified and detailed models; Contourplot of detailed model with diagonal crack in connection and cracks in connecting wall (last increment)

CONCLUSION

Three connection types have been tested at the Materials Testing and Research Institute (MPA Karlsruhe). The design of the test samples and the interpretation of data were accompanied by SMP Ingenieure im Bauwesen GmbH who has also done the numerical simulations. The wall to wall connection was tested with two different stud configurations. The first one was tested in bending dominated loading only and the second type with more studs in the connection region was tested in bending and separately in shear. The resistance of both connection types was sufficient to resist the nominal bending/shear resistance of the connecting wall. However the connection with few studs showed intense cracking, concrete spalling and a steel plate buckling at high loading rates at the bending test. Somewhat surprising was the occurrence of shear cracks in the connecting wall far before the nominal resistance acc. to different codes. Furthermore the nominal shear resistance varies largely between the different codes. The shear resistance of the connecting wall was the limiting factor in all wall to wall tests – even at the bending dominated tests.

For the numerical simulations a complex modelling approach was selected were all components of the SC type of construction are modelled explicitly. In a more detailed variant the interaction between tie bars/studs and concrete was modelled with contact surfaces. This modelling approach showed good agreement to the tests with a deviation of the ultimate load of less than 12.5%. However the early cracking in shear and the failure of the connecting wall in shear at the bending dominated tests could not be predicted with the numerical simulations.

All findings were transferred to the project partners within the SCIENCE research project in order to develop and implement regulations into the Eurocode design concept. These regulations were not part of this paper and can be obtained soon with the final report.

REFERENCES

Bassam, Burgan (2015) “European Research into Composite Steel - Concrete Structures” *SMiRT23*, Manchester, United Kingdom - August 10-14, 2015

Bassam, Burgan (submitted) Final report: SC for Industrial, Energy and Nuclear Construction Efficiency (SCIENCE), Research Programme of the Research Fund for Coal and Steel Technical Group: TGS8

Hrynyk, T.D., Vecchio, F.J. (2015) “The Influence Of Lateral Expansions On The Response Of Steel-Concrete Composite Structures” *SMiRT23*, Manchester, United Kingdom - August 10-14, 2015

Varma, A.H., Sener, K.C., Zhang, K., Coogler, K., Malushte, S.R. (2011) “Out-of-Plane Shear Behavior of SC Composite Structures”, *SMiRT21*, New Delhi, India - November 6-11, 2011

Kurt, E.G., Varma, A.H. (2015) “Preliminary Investigation of Steel-Plate Composite (SC) Wall-to Concrete Basemat Anchorage Connections”, *SMiRT23*, Manchester, United Kingdom - August 10-14, 2015

Seo, J., Varma, A.H. (2015) “Behaviour and Design of Corner or L-Joints in SC Walls”, *SMiRT23*, Manchester, United Kingdom - August 10-14, 2015

AISC N690-12 (2012) incl. supplement AISC N690s1-15. Specification for Safety-Related Steel Structures for Nuclear Facilities. American Institute of Steel Construction (AISC).

KEPIC-SNG (2009) Korea Electric Power Industry Code: Specification for Safety-Related Steel Plate Concrete Structures for Nuclear Facilities

JEAC 4618 (2009). Japan Electric Association: Technical Code for Aseismic Design of Steel Plate Reinforced Concrete Structures. Translated into English in January 2012 by the Obayashi Corporation

EN 1994-1-1 (2004) Eurocode 4: Design of composite steel and concrete structures - Part 1-1: General rules and rules for buildings