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Longitudinal shear capacity of steel deck concrete composite slabs after 35 years in service – In situ tests at Zurich Airport

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ABSTRACT: This paper presents an experimental study on composite slabs with profiled steel sheeting (Holorib 51) performed at Zurich Airport in conjunction with the conversion of Dock B. The structure was built in 1974 with composite floor slabs, a steel frame with stub-girder beams and hot-rolled steel columns as supporting structure. The experimental investigation focused on the bond behavior of the composite slabs and the influence of the 35 years service life. Ten large-scale tests were carried out on simple beams with different shear span length to evaluate the shear capacity. In addition, tests on two-span beams were performed. The slab beams were cut from the slab and tested in situ. The data obtained by these tests enabled the assessment of the slabs. The paper presents the results of the tests on simple and continuous beams and the assessment of the results with common design methods.

1 INTRODUCTION

Composite slabs are widely used all over the world for more than 50 years. The use of profiled steel sheets in combination with a concrete layer results in a good solution for the construction of building floors. Steel deck concrete composite slabs gain structural benefits, allow fast erection and are lightweight. Two design methods, namely the m-k-method (Porter & Ekberg 1976) and the partial connection method (Bode & Sauerborn 1992), are mainly used for the verification of composite slabs. These methods base on previous investigations that experimentally analyze the load-bearing behavior of the slabs and in particular the longitudinal shear bond strength that is often crucial for the capacity of the slabs. The tests analyze the longitudinal shear bond behavior of new slabs, usually casted for the tests, in laboratories. The reconstruction of Terminal B of Zurich airport allows experimentally analyzing the longitudinal shear bond behavior of composite slabs with profiled steel sheets after 35 years in service.

Terminal B at Zurich Airport was built in 1974. It will be converted to implement the Schengen agreement with the European Union. As a precondition to reuse the existing structure it had to be shown that the structure still meets the safety and serviceability requirements in particular the longitudinal shear capacity of the composite slabs after 35 years in service. Hence, destructive large scale tests were performed in situ on the composite slabs. The test results on simple beams were assessed with the m-k-method and the partial shear connection method according to EN 1993-1-1 (2004). The test results show that the longitudinal shear strength is still in the range of the initial shear strength.

This paper first describes the experimental program for the in situ tests on steel deck concrete composite slabs at Zurich airport. Next, the test results including load-deflection and load-slip history records are presented. Finally, the shear bond parameters required for the m-k-method and the partial shear connection theory are determined based on the simple beam test results.

2 EXPERIMENTAL PROGRAM

2.1 Test specimen

The longitudinal shear bond behavior of the steel deck concrete composite slabs was experimentally analyzed with destructive in situ tests at Terminal B of Zurich airport. Destructive tests became possible because the airside part of the terminal was demolished and the remaining part (landside) was undergoing major changes due to new requirements of use. The two-storey building of Terminal B at Zurich airport consisted of HEM 360 columns and virendeel girders made of two HEA 160 profiles as flanges and a steel plate as a web. Secondary IPE 360 beams constituted the supports of the composite floor slab with profiled steel sheetings. The profiled steel sheetings were fixed to the top flanges of the secondary beams using powder activated fasteners. The composite slabs did not have any kind of additional end anchorages. The composite slabs consisted of Holorib HR51 trapezoidal steel sheetings with a nominal thickness of 0.91 mm and a height of the dovetail rib of 51 mm (spacing of 150 mm), a layer of structural concrete of 120 mm and a reinforcement mesh d = 4mm/150mm. A non-structural screed with a thickness of 120 mm was placed at top separated from the structural concrete by a coated paper foil. The cross section of the composite slab is given in Figure 1.

The actual material properties of the profiled steel sheets and the structural concrete were determined. The compressive and the tensile strength of the concrete were investigated with five and four respectively small cylinders (mean height h = 112 mm, mean diameter d = 44.7 mm) bored from the slab with a diamond core drill. The mean value of the concrete compressive strength was 54.5 N/mm^2 (COV 0.063). The mean value of the concrete tensile strength was 3.86 N/mm^2 (COV 0.114). The yield strength and the ultimate tensile strength of the steel sheet were determined with four tensile material coupon tests according to DIN EN ISO 6892. The coupons were cut from the steel sheeting of the composite slab and had a shape according to EN 50125. The mean value of the ultimate tensile strength was 339.1 N/mm^2 (COV 0.012); the mean value of the yield strength was 273.9 N/mm^2 (COV 0.034).

To investigate the longitudinal shear capacity of the composite slabs, ten full-scale tests were performed on simple beams with different shear span lengths. The span of the simple beams was 2.58 m. The bonding between the Holorib 51 steel deck and the concrete was only provided by the rolled dovetail shape of the Holorib 51 and had no embossments. Two tests on two-span beams were additionally performed to experimentally analyze the continuous action and the moment-curvature behaviour of the composite slab without reinforcement at the top of the slab.

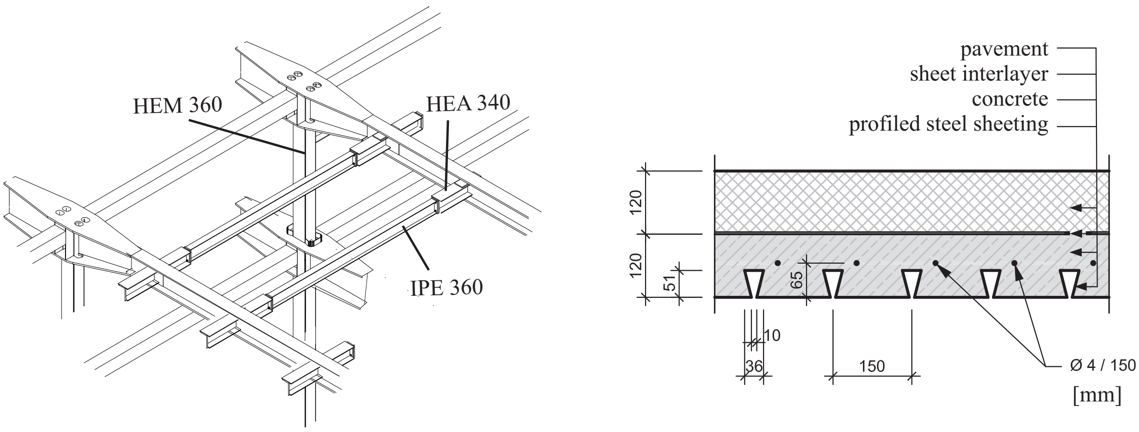

Figure 1. Detail of the steel structure of Dock B at Zurich airport including steel frame structure with stub-girders and secondary beams (IPE 360 profiles) (left) and cross section of the composite floor slab with profiled steel sheeting, concrete and pavement (right).

2.2 Test setup and measurement

The non-structural screed was removed before testing. The same method and construction site equipment intended for the refurbishment of the part of the terminal designated for reuse were used for the removal. The test specimens were produced out of an inner part of the lower floor slab at the airside part of Terminal B. The specimens were diamond cut from the slab and left in situ for testing as either simple or continuous beams supported by the secondary beams. The nominal width of the test specimens was 850 mm. To fix a testing frame to the secondary beams holes with a diameter of 50 mm were drilled adjacent to the secondary beams. The testing frame mainly consisted of two U-profiles and I-profiles (Fig. 3). The testing frame had to be removed without any crane in a very short time because the entire testing program had to be carried out within two weeks to avoid any interference with the ongoing reconstruction process. The frame was fixed to the secondary beams using high strength steel rods passing through the holes.

The test specimens were tested as four-point and three-point bending tests. The loads were applied by hydraulic load jacks, which could be positioned at different shear spans. The load was applied manually using a hand pump. The load was measured using the servo hydraulic pressure. The mid-span deflection and the deflection close to the jacks were measured using LVDTs. LVDTs were also used to measure the slip at a distance of approximately 400 mm from the supports, cp. Figure 2 to 4. Details of the test setup and measurement are given in Klippel, Knobloch & Fontana (2010).

2.3 Test procedure

2.3.1 Tests on simple beams

Ten full-scale tests (including one pre-test) were carried out as three- point and four-point bending tests on simple beams with a span between the supports (IPE secondary beams) of 2.58m. The distance from the supports to the point loads was tantamount to the shear span length of the bending tests. The nominal distance of the three-point bending tests was L/2 = 1290mm (equal to half span). The four-point bending tests were performed with different shear span lengths. The nominal shear span lengths were L/3 = 860mm, L/4 = 645mm and L/8 = 322.5mm. The shortest shear span length was roughly the same as three times the depth of the composite slab. Figure 3 shows the frame for a test with $L_s = L/2$ (left side) and $L_s = L/4$ (right side). The rules for laboratory tests given in EN 1994-1-1 (2004) were followed as close as possible. The test specimens were first subjected to three to five loading cycles with the maximum value equal to approximately 40% of the estimated ultimate load. The composite floor slab had already been loaded to many loading cycles during its service life. Thus, it seemed not be necessary to perform more loading cycles as it is recommended for laboratory tests on new composite slabs. After performing several loading cycles, the specimens were loaded until first slip occurred. Then, the specimens were loaded either to failure or until very large deflections prohibited further progress.

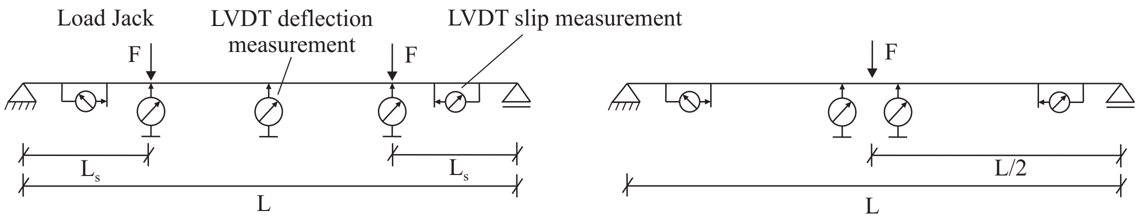

Figure 2. Loading scheme for simple beam tests: 4-point-bending (left) and 3-point-bending test (right).

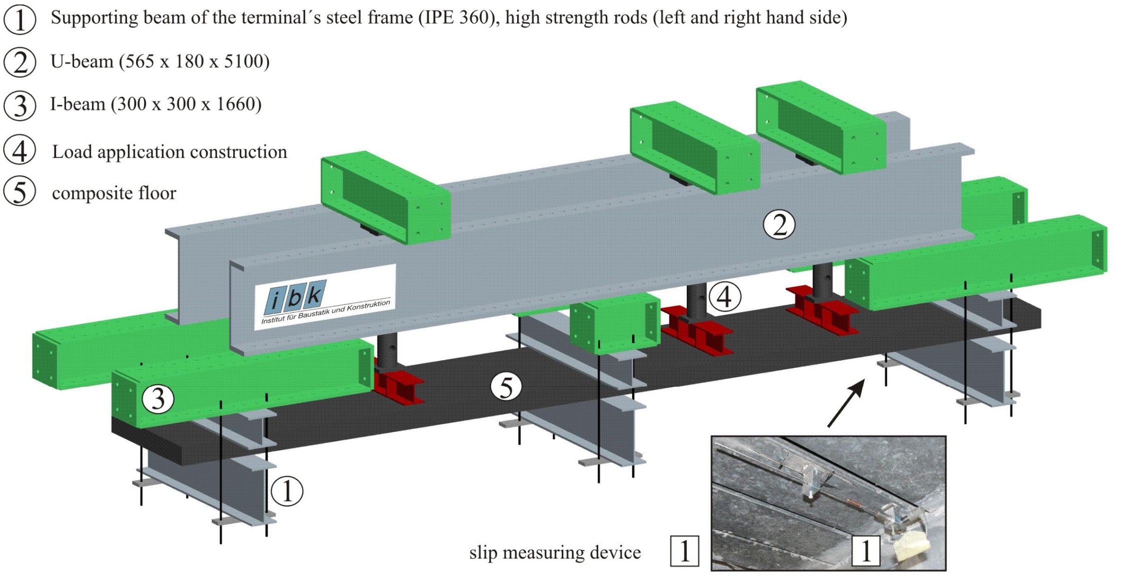

Figure 3. Testing device and bottom view of the slab with measurement.

2.3.2 Tests on continuous beams

In addition to the simple beam tests, two tests on two-span beams were carried out to investigate the load bearing behaviour of continuous beams. The tests were performed as three-point-bending tests at both spans with a shear span length $L_s = L/2$ (Figure 4). Both spans had a length of L = 2.58 m. The procedure of the two-span beam tests was similar to the simple beam tests.

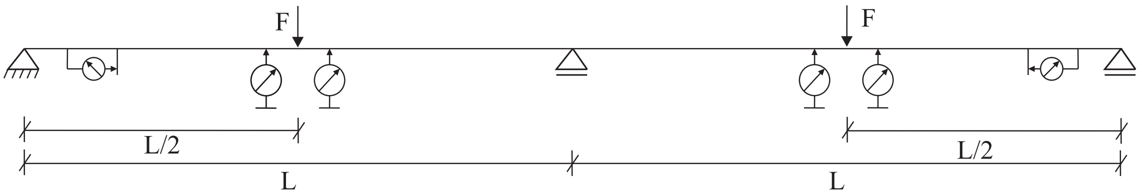

Figure 4. Loading scheme for tests on continuous beams (shear span length in both spans $L_s = L/2$).

2.4 Test results

2.4.1 Tests on simple beams

The failure of all test specimens (even the test specimens with short shear spans) was governed by longitudinal shear failure. No vertical shear failure was observed. Figure 5 shows the load-deflection (left) and load-slip curves (right) of the simple beam test specimens as examples. The deflection is given as the vertical deflection at mid-span and the slip is given as the mean value of both slip measurements. The specimens showed almost linear-elastic behavior until the first distinct slip between the steel sheeting and the concrete was noticed. After the first slip occurred the load dropped down abruptly and increased again for most of the specimens. The behavior of the specimens was classified ductile if the ratio F_u/F_s between the remaining ultimate load (maximum load after load drop caused by slip occurrence) and the load at first slip exceeded 1.1. Otherwise the behavior of the specimens was classified non-ductile. According to this classification six of nine test specimens showed a ductile behavior. Table 1 gives the load at first distinct slip, the ultimate load, and the corresponding deflections.

The load-deflection behavior was similar for test specimens with equal nominal shear span lengths. However, the ultimate load of the two three-point bending test specimens (sb 7 and sb 10) is highly different. The remaining ultimate load of sb 10 is 43% higher than the load of sb 7. For most test specimens powder activated fasteners were only found at the inner support. However, test specimen sb 10 possessed fasteners on both supports which might be the reason for the higher resistance.

Table 1. Test results for different shear span length (load F per jack).

L_s	*L/8*		*L/4*			*L/3*		*L/2*	
Test specimen	sb 6	sb 8	sb 3	sb 5	sb 9	sb 2	sb 4	sb 7	sb 10
Load F_s [kN] at first slip	36.1	43.6	40.8	45.4	41.9	27.2	22.9	47.0	80.4
Max load F_u [kN]	50.3	52.7	49.0	45.4	45.0	32.3	25.5	47.0	80.4
Deflection w_u [mm]	11.1	14.6	21.9	24.4	16.0	42.0	26.2	13.3	35.9

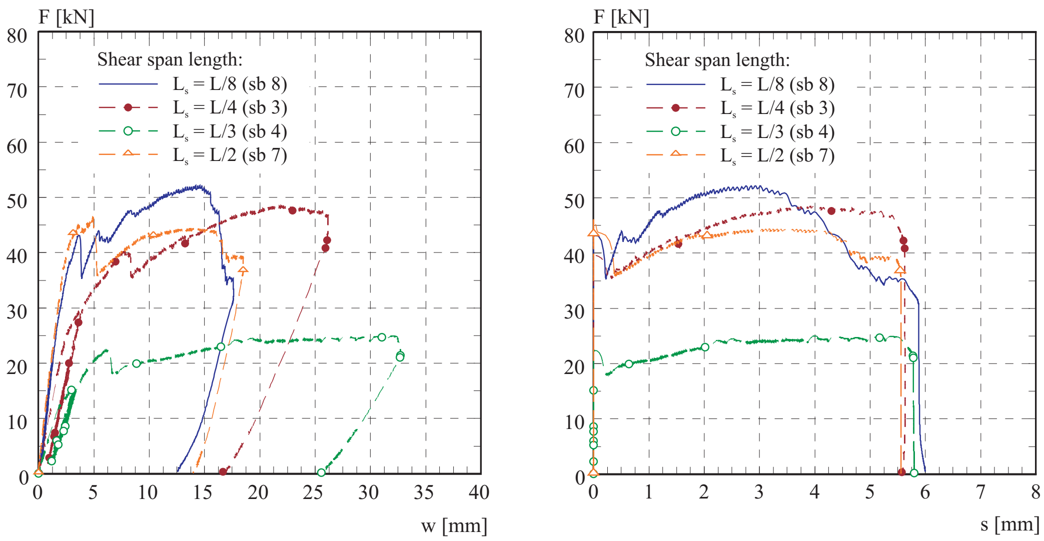

Figure 5. Load-deflection (left) and load-slip behavior (right) for different shear span lengths L_s (applied load F is given per jack).

2.4.2 Tests on continuous beams

Figure 6 shows the load-deflection (left) and load-slip curves (right) of the two-span beam test specimens. The main results of the two-span beam tests are summarized in Table 2. The two-span beams did not show a ductile behaviour according to EN 1994-1-1 (2004). The two tests on two-span beams showed different load-bearing behaviour. The load-deflection curves of the two spans of test specimen cb 1 showed almost equal behaviour. At a certain load level the slip at both outer supports occurred and in particular the deflection and the slip in span 1 increased until a longitudinal shear failure occurred (Figure. 6). However, the load-deflection behaviour of the two spans of cb 2 was different. The deflection of span 1 of test specimen cb 2 was much larger than the deflection of the span 2 and the failure was governed by the failure of span 1. A crack at the upper side of the slab occurred at a load level of 21.8kN at test specimen cb 2. The applied force could be raised in both areas until the first slip occurred in span 1 (Figure. 6). The slip occurred only in span 1, hence, the applied load was especially redistributed to this span. Failure was governed by longitudinal shear failure in span 1.

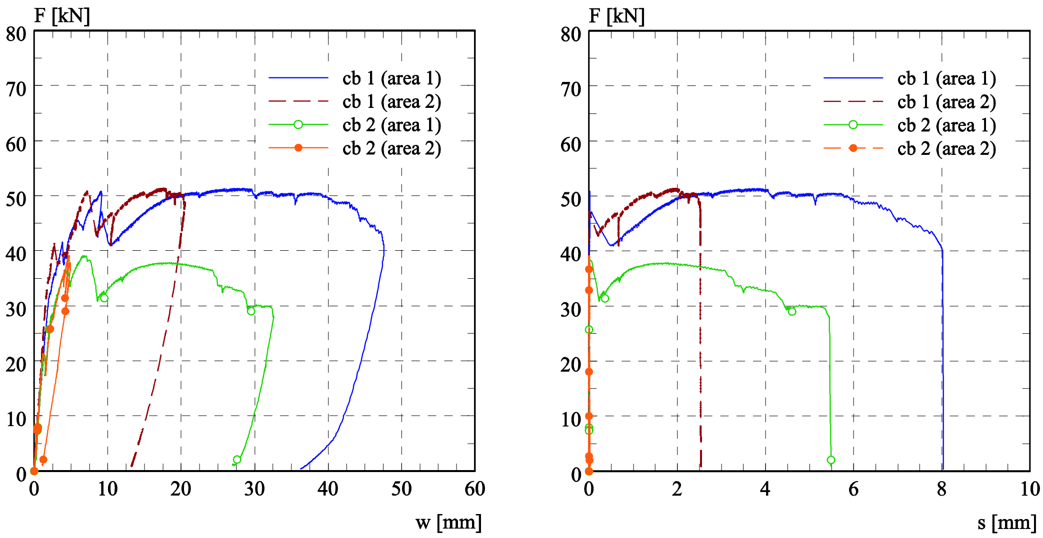

Figure 6. Load F per jack versus deflection (left) and load F per jack versus slip (right) for the tests on continuous beams.

Table 2. Test results for the tests on continuous beams (load F per jack).

Test specimen	*cb 1*		*cb 2*	
	span 1	*span 2*	*span 1*	*span 2*
Load F_s [kN] at first slip	46.1		39.0	
Max load F_u [kN]	52.0		39.0	
Deflection w_u [mm]	20.0	22.0	17.5	4.9

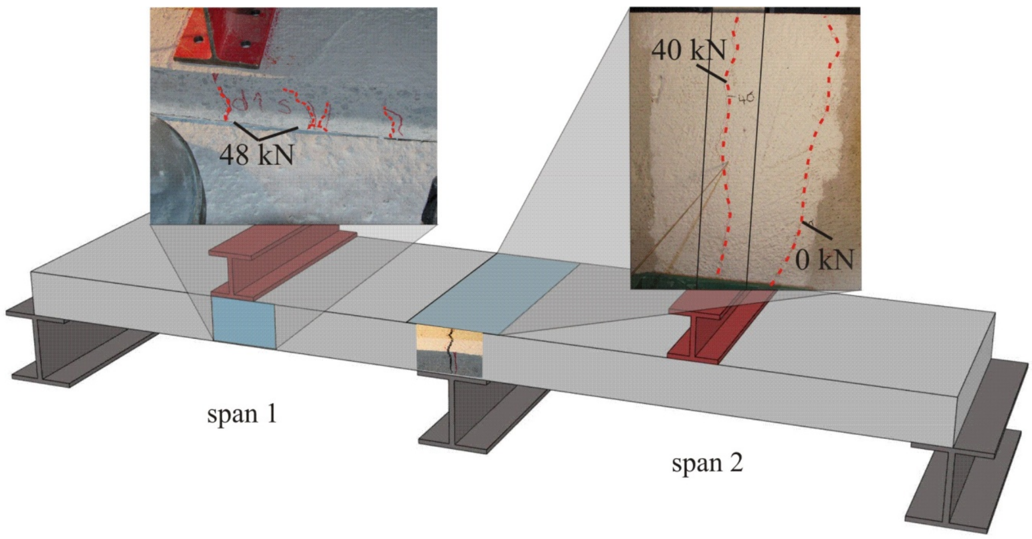

Figure 7. Crack pattern of the two-span beam test cb 1.

3 ASSESSMENT OF THE TEST RESULTS

The test results were assessed on the basis of simple analytical models, namely the semi-empirical m-k and the partial shear connection method. Steel deck concrete composite slabs show three typical failure modes in function of the shear span length. For very short shear spans vertical shear failure may be observed while for very long shear spans bending failure is observed. However, the capacity of the most end spans of composite slabs in buildings is governed by longitudinal shear failure typical for intermediate shear spans.

3.1 m-k-method

The m-k-method is based on a test program on four-point bending tests with at least six test specimens with two or more different shear span lengths. By means of a linear regression analysis the empirical factors m and k are determined and the longitudinal shear resistance is assessed according to Eq. (1).

$$V_{l,Rd} = \frac{b \cdot d}{\gamma_{VS}} \cdot \left(\frac{m \cdot A_p}{b \cdot L_s} + k \right) \tag{1}$$

Figure 8 (left) shows the vertical shear capacity of the simple beam tests $V_{l,Rd}$ (divided by the width b and distance between the centroidal axis of the steel sheeting and the upper side of the slab d_p) as a function of the reciprocal value of the shear span lengths L_s (multiplied by the width b of the slab and divided by the cross section of the sheeting A_p) considering actual geometrical and material properties. For the assessment of the characteristic values m and k of the method the vertical shear capacity of all test specimens classified as non-ductile were first reduced by 20%. Secondly, the minimum capacity of each group of shear span lengths considering test specimens classified as both ductile and non-ductile was reduced by 10% for simplicity, and thirdly, a linear regression analysis was performed. Based on the m-k-method a design value of the longitudinal shear resistance of $V_{l,Rd} = 25.6$ kN was calculated for the composite slabs from the results of the in situ tests.

3.2 Partial shear connection method

Composite slabs with typical shear span length usually fail by exceeding their longitudinal shear resistance before reaching their plastic bending moment capacity. As an alternative to the empirical m-k-method the test results were assessed on the basis of the partial shear connection method which is based on the mechanical model of partial shear connection (Bode & Sauerborn 1992). A detailed description of the procedure is given in Annex E of the EN 1994-1-1 (2004), for example. The characteristic bond strength τ_u of the composite floor slab may be obtained by reducing the minimum value obtained from all tests (considering specimens classified as both ductile and non-ductile) by 10 % for simplification. The design shear strength $\tau_{u,Rd}$ is calculated by dividing the characteristic value by the partial safety coefficient.

Figure 8 (right) shows the experimentally determined ultimate bending moments (dots) as well as the design bending resistance M_{Rd} calculated with the partial shear connection method (continuous line) as a function of the normalized shear span length L_{sx}/L_{sf}. Only for a shear length exceeding L_{sf} the full plastic moment capacity of the slab is reached. The bending resistance is based on the design shear strength as well as on the actual material and geometry data considering partial safety factors according to EN 1994-1-1 (2004) For the design of the slabs with partial shear connection it has to be shown that the design bending moment M_{Ed} does not exceed the design resistance M_{Rd}. Figure 8 (right) shows the bending moment M_{Ed} for a uniformly distributed load (dotted line). The bending moment for a uniformly distributed load M_{Ed} equals the bending resistance M_{Rd} for a shear resistance $V_{l,Rd} = 27.0$ kN according to the partial shear connection method. The shear resistance obtained by the partial shear connection method is slightly higher compared to the value obtained by the m–k-method. This is in accordance with Sauerborn (1995).

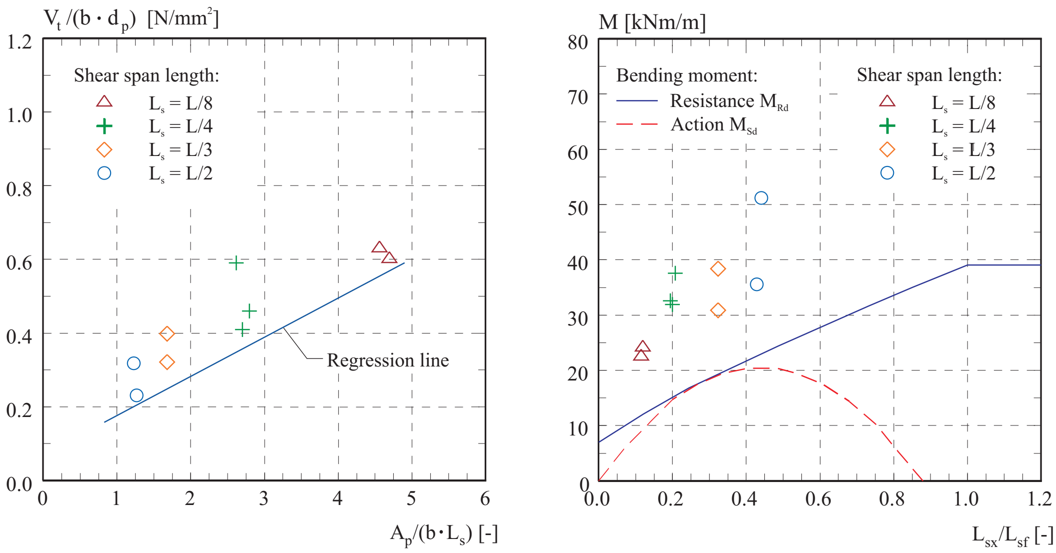

Figure 8. Evaluation of the test results with m-k-method (left) and partial shear connection theory (right).

4 CONCLUSION

The behaviour of a steel deck composite slab with profiled steel sheeting after 35 years in service has been experimentally investigated by large scale in situ tests. In all tests, the slabs showed longitudinal shear failure (bond between profiled steel sheeting and concrete). No vertical shear failure was noticed even for the tests with short shear span length of about three times the height of the slab.

The tests were assessed by the m-k and the partial shear connection method for composite slabs. It was found that the shear resistance obtained by the partial shear connection theory was slightly higher compared to the value obtained by the m-k-method. The results show that the profiled steel sheeting after 35 years in use still features good longitudinal shear strength and is fit for reuse as slab of Dock B of Zurich airport.

5 REFERENCES

Bode H., Sauerborn I. 1992. Zur Bemessung von Verbunddecken nach der Teilverbundtheorie. Stahlbau, 61(8): 241-250.

EN 1994-1-1. 2004. Bemessung und Konstruktion von Verbundtragwerken aus Stahl und Beton – Teil 1: Allgemeine Bemessungsregeln und Anwendungsregeln für den Hochbau, CEN, Brüssel.

Klippel M., Knobloch M., Fontana M. 2010. Umbau Dock B am Flughafen Zürich – Belastungsversuche an Holorib-Blechverbunddecken, Versuchsbericht mit Auswertung. Institut für Baustatik und Konstruktion (IBK), ETH Zürich. IBK Bericht Nr. 325. Birkhäuser Verlag Basel.

Porter M. L., Ekberg, C. E. 1976. Design Recommendations for Steel Deck Floor Slabs. *Journal of the Structural Division*, ASCE, 102(11): 2121-2136.

Sauerborn I. 1995. Zur Grenztragfähigkeit von durchlaufenden Verbunddecken, Universität Kaiserslautern.